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Proceedings of the American Society of Civil Engineers

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Proceedings of the American Society of Civil Engineers

THE PEORIA RECHARGE PIT: ITS DEVELOPMENT AND RESULTS

Max Suter,[1] M. ASCE
(Proc. Paper 1102)

SYNOPSIS

Research on artificial recharge was done in Peoria, Illinois by the State Water Survey to find methods for overcoming the losses in ground water storage due to overpumpage. A method was found to obtain the high rate of inflow of from 23 to 27 mgd per acre. Many types of hydrologic, chemical, and bacteriological observations were made. Some of the relations found cannot yet be explained.

INTRODUCTION

In Peoria, Illinois, within an area of 12.4 square miles, about 50 mgd are pumped from glacial till and alluvium. This pumpage, being more than the natural inflow, has caused a gradual lowering of the ground water level. The complicated geologic and hydrologic conditions are described in detail in Bulletin 39 of the Illinois State Water Survey.[2] The main factors of importance to this report are discussed here.

The pumpage is done from three well fields which are more or less separated by ridges in the bedrock. The term "well field" designates an area within which a continuous interference can be traced from well to well. About 50 per cent of the pumpage is for municipal use, the remainder is industrial pumpage. In the period 1934-1942, the lowering of the ground water level was about 2 feet per year. Since then, partly as a result of measures taken to conserve water and temporary raises from high river stages, it is about 0.6 of a foot per year and only 0.1 of a foot in the last two years. The ground water level in the main pumping area is now as much as 25 feet below pool level of the Illinois River. In some areas of high bedrock only 10 feet of saturated aquifer remain. From determinations of the volume of the

Note: Discussion open until April 1, 1957. Paper 1102 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 82, IR 3, November, 1956.

1. Head, Eng. Research Subdivision, Illinois State Water Survey, Urbana, Ill.
2. "Ground Water in the Peoria Region" by Leland Horberg, Max Suter, T. E. Larson, Illinois State Water Survey Bulletin 39, 1950.

pumped-out-strata, it was found that the overpumpage is in the range of 8 to 10 mgd. Practically all of the natural inflow comes from infiltered rainfall in the surrounding territory. The Illinois River is silted to such an extent that it contributes only traces when at pool level, although during flood periods some recharge occurs from it. The conservation measures mentioned consisted of using river water instead of ground water for condensers, and of the introduction of recirculating systems with cooling towers. Although the reduction of industrial pumpage amounted to around 15 mgd, its effect was later partially voided by an increase in municipal pumpage with a peak of 13 mgd. As the overpumpage could not be eliminated by conservation measures, studies were started to find out if artificial recharge could compensate for any deficit of natural inflow and also whether or not a reserve could be built up for future needs.

The Problem

The problem in this study was, then, to find a way to recharge a large amount of water through a small surface area, as plots for infiltration operations are scarce in the industrial district, and recharge through wells would have required a filter plant to treat the river water. Previous tests with recharge wells using well water showed that these wells clog within three months by iron deposits.

Preliminary Studies

Inasmuch as the demand is for cold ground water of a temperature of not more than 60°F, it was realized that recharge could be done only during the colder season. Studies[3] had shown that, in Peoria, the river has a temperature of 60°F or less for an average of 209.8 days, with a minimum of 181 days and a maximum of 236 days.

Six months were assumed as an average time of recharge operation. Previous tests in an abandoned gravel pit about one-half mile south of the proposed location showed that a recharge rate of 37 mgd per acre could be obtained. Test drilling showed that at the proposed location the gravel had 2.1 per cent of material passing the 200 mesh screen compared to 0.5 per cent at the gravel pit. A pit was, therefore, designed with the hope that a rate of inflow of 20 mgd per acre could be attained.

The State of Illinois is not willing to go into the water business for any specific community, but only to help in the research problems connected with it. State funds did provide for the construction of a small pit estimated to recharge 0.3 mgd. The local interests desired a larger pit which would recharge an average of 1 mgd over the year or 2 mgd over the six-month operating season. In consequence the local industries collected, through the Peoria Association of Commerce, some $75,000.00 to enable the construction of the larger pit.

3. "Temperature and Turbidity of Some River Waters in Illinois" by Max Suter, Illinois State Water Survey Report of Investigation No. 1, 1948.

Description of Pit

The pit is constructed on the river side of the 400 x 165 feet lot on which the State Water Survey has its Peoria Hydraulic Laboratory.

Figure 1 shows a plan and a longitudinal section of the recharge pit.

The water is taken from the river at a sheet piling intake along the "Harbor Line." The 16 inch pipe is 4 feet below pool level, which is at elevation 440.0 MSL, and screened by a bar screen with 2 inch openings. A valve is located directly back of the sheet piling in a manhole. The water passes under the railroad to the control tower which is a 16 foot internal diameter concrete caisson. A 45° horizontal Ell at the control tower directs the water towards the chlorine injector and gives the water a rotating movement. The chlorinator has a capacity of up to 1000 lbs Chlorine per day. The chlorinated water passes through a vertical 12 inch Everdur well screen with 1/8 inch slots and then through a 12.125" x 9.352" Venturi tube connected by compressed air to a recording and integrating meter. A 12 inch transite pipeline with a valve conducts the water to the pit, where the water is discharged vertically through a 90° Ell.

The pit itself has a 40' x 62-1/2' essentially rectangular bottom at elevation 430.0. Up to elevation 442.0 it has slopes of 1:2. The bottom and slopes to elevation 442.0 are covered with a 6 inch layer of sand or pea gravel which acts as a filter. From elevation 442.0 to the top of the surrounding levee, at elevation 460.0, the slopes are 1:1-1/2. These slopes are in grass.

The inflow is by gravity and not by pumping. The amount of inflow is, therefore, regulated by the infiltration capacity of the pit, the loss of head in the inflow structures, and the river stages.

The effect of the recharge on the ground water is charted by 11 recorder wells surrounding the pit at distances of 88 feet to three-quarters of a mile from the pit. Nine industrial wells give information on the changes in water levels and temperatures. Weekly temperature readings are also taken in the recording wells at various depths with an electric conductivity thermometer.

Water elevations in the river, control tower, and pit are measured hourly to the nearest one-tenth of a foot. The fixed gages are painted alternatingly red and white every foot for easy reading.

Chemical and bacteriological tests on the river water and in a well with a 20 gpm pump at the laboratory are made four times weekly.

The pit is now cleaned once a year just prior to starting a seasonal operation.

Operation of Pit

The pit is now, Winter 1955-56, in its fifth season of operation. Some changes were made in each seasons operation to evaluate the factors involved in producing an efficient, economic, high rate recharge. Research of these factors is the principle aim and not the amount of recharge.

In the first three seasons clean sand with an effective size of 0.3 to 0.4 mm and a uniformity coefficient of 2.0 was used as a 6 inch deep filter. This sand clogged rapidly from the silt in the river water, at a rate averaging a 60 per cent loss of inflow in six months.

During the first season, 1951-52, the operation of the pit was shut down whenever the turbidity reached 100 ppm. This caused much loss of time,

especially in freezing weather when the pit froze during the shutdown and could not be put back into operation for some time.

Even with the shutdowns regulated by turbidity limits, the reduction in inflow was so great that the entire filter sand was twice removed and replaced by new sand. The first shutdown also revealed that the bottom layer of original ground was very compacted from the bulldozer used during the construction. Blasting of this bottom increased the recharge. In this first season there were four periods of high water and the rate of recharge increased during these periods. However, this increase is not in correspondence to the height of the flood, probably due to increases in silt. The average rate of inflow was 1.78 mgd for the first season's operating period.

During the second season the pit operated continuously and only a minor period of high water occurred with a peak of only 2.8 feet above pool. In this season the terminal rate of inflow was 43 per cent of the starting rate. The average rate of inflow was only 1.03 mgd.

In the third season the operation was also continuous. Towards the end of the season some attempts were made to clean the sand with a swimming pool type suction cleaner. It took many adjustments before a sliding base could be made so that the cleaner would remove only the silt and not the sand. By that time the season was over and the net result was discouraging as the final rate of inflow was only 35 per cent of the original rate. The average rate of inflow was 1.05 mgd.

The fourth season, for a radical change, a clean pea gravel with a grain size of 3.4 to 9.3 mm and a uniformity coefficient of 1.35 was used as a trial for a filter material. An increase in the rate of recharge was soon noted. An average seasonal rate of 3.08 mgd was obtained. The loss in rate of recharge was also greatly reduced, the final rate being 88 per cent of the recharge rate at the start. There were some predictions that this pea gravel would not filter out the silt and that the original ground would be clogged. However from tests on samples from the pit it was found that a silt layer formed over the pea gravel, with the top layer of the gravel containing most of the silt, and the bottom layer still clean after one month of operation. Later the bottom layer was found to contain some silt.

Cleaning of this gravel could not be done with the cleaner as developed for use on sand because the cleaner sank into the gravel. A larger base had to be built and rebuilt. The cleaner is still not working satisfactorily, especially when pulled up the side slopes. However it does clean the bottom gravel to a depth of one inch.

The fifth, present season, uses the same pea gravel filter and its rate of inflow started with an average inflow of 3.51 mgd without the effect of any high river levels. Efforts are being made to maintain or improve this rate throughout the entire operating season.

A typical summary of various factors are shown in Fig. 2 for the season 1953-54. They indicate that turbidities are practically independent of river stages, but the inflow and well levels rise and fall with periods of higher water. The recharge was not sufficient to entirely overcome the lowering effect on the ground water due to pumpage as shown by ground water elevations.

Studies on Results

The relation between volume, water surface area, wetted surface area, and surface areas at different depths of water in the pit are shown in Fig. 3.

The difference between river and pit elevation depends on the loss of head in the connecting system of pipes, valves, bends, screens, and Venturi tube. This loss of head cannot be accurately calculated because the loss not only depends on the amount of flow, but also to a greater extent on the conditions of the screens in use.

Plotting flow against loss of head, Fig. 4, shows a great many points scattered over the area. However, these points can be enclosed by an upper enveloping curve which gives the minimum loss of head necessary for a certain flow. It indicates and can be used to check the condition of the screens cleanliness.

With the low inflows, around 1 mgd, obtained in the first three seasons, the loss of head was about 1 foot and the surface area 1/6 of an acre. With the higher inflow of 3 to 3.5 mgd, the loss of head is around 3.5 feet which gives surface area of about 1/8 of an acre. The rate of recharge is, then, from 6 mgd per acre with sand to 23-27 mgd per acre with pea gravel as filter. This corresponds to an inflow of 70 to 82 feet per day. These recharge rates are much greater than those of other recharge operations noted in literature. American installations very seldom exceed 0.5 mgd per acre (1.5 ft per day) and none is reported over 1.0 mgd per acre. A Swedish installation (Katrineholm)[4] is reported to have an inflow of 16 mgd per acre (49 ft per day).

Since this paper was submitted, a pit was built by a private company in Peoria, using the results of the State Water Survey investigations, and this pit has the surprisingly high inflow of 72 mgd per acre or 220 feet per day.

The high rates obtained at Peoria require, therefore, some investigation for explanations of their occurrence.

Model Tests

As frequent changes in pit construction is expensive and also since underground flow conditions in a field pit are difficult to study in detail, recourse was made to a model in which shape of pit and flow levels could be changed in a wide range.

This model was designed to represent a 45° section of the pit, or a centerline to diagonal section. The centerline strip is 13 feet long and the total depth of sand is 5 feet. On the centerline are 81 gages in 9 rows and on the diagonal there are 72 gages in 8 rows. The gages are 8 mm glass tubing mounted on a special board with a clock. Momentary conditions with respect to time are recorded by camera.

Test data thus far show that the main flow occurs through the sloping walls and that the total flow through the bottom increases only with the square root of its size. The flow increases also with the depth of water in the pit and with the total drop between level in pit and original ground water level.

The exact data have to be reserved for a later publication when more verification tests have been made.

The model showed a high efficiency pit should have most of its wetted area in the sloping sides, should have the maximum possible of water depth, and should be located as high as possible above the ground water level. There

4. "Artificial replenishment of Underground Water" by Victor Jansa, 2nd Congress International Water Supply Assoc., 1952, Paris, p. 163.

are some good reasons for this behavior. A pit recharging at equilibrium flow can recharge only as much as can flow away from it into the surrounding strata. The inflow into the ground depends on the wet area, the characteristics of the top soil layer and the effect of silting. The flow-away capacity in the ground depends on the flow area available, the permeability of the soil and the slope of the ground water surface.

Water entering the soil through the side slopes of the pit has a direct streamline path for flowing away through the strata. The area available is proportional to the perimeter at the water surface.

Water entering the soil through the bottom of the pit flows with sharply bent streamlines and has, as possible flow-away area, only that which is left between the flow-away from the slopes and the original ground water level. This flow area grows lineally with the perimeter of the pit whereas the bottom area grows (in pits of nearly square shapes) with the square of the perimeter. It is possible that with small bottoms more flow-away capacity is available than what can be fed by the inflow through the bottom. With large bottoms more water could enter through the bottom if the flow-away area would also change with the square of the perimeter. An elongated shape of the pit can provide a favorable ratio of these flows even in large pits.

Field Observations

Screening

Screening is used to keep larger floating particles out of the pit. A bar screen with 2 inch openings at the river keeps most of the debris and larger fish out. Smaller fish which come into the control tower are killed by the chlorine. They, as well as leaves and grasses, can clog the fine screen which is difficult to clean when the velocity through its openings is higher than 0.6 foot per second. With these higher velocities clogged screens can be cleaned only by shutting off the flow for a short interval.

Filtering

Filtering was provided as a final safety measure and to eliminate clogging of the original ground material. Even when only 6" deep, the sand used acted as a rough sand filter. The inflow was held in the neighborhood of 1 to 1.5 mgd and reduced rapidly as silt accumulated.

The results with the use of pea gravel are pleasantly surprising. The average inflow was greatly increased, 3 to 3.5 mgd, and silt accumulation reduced the inflow very little.

A 1" silt layer over the sand filter reduced the inflow 60 per cent, whereas a similar layer over the pea gravel reduced it only 12 per cent with similar flow-away conditions.

There are certain difficulties in explaining the different behavior of sand and gravel. However, it is certain that the material with the least permeability governs the inflow. The clean sand has a permeability of 9000 Meinzer units[5] and the silted sand less than one-half thereof. The ground has a

5. Meinzer Unit: Flow of water in gallons per day through a cross-sectional area of 1 sq. ft. under a hydraulic gradient of 100 per cent at a temperature of 60°F.

permeability of 8000 Meinzer units, whereas the clean pea gravel has a permeability of 155,000 Meinzer units. The permeability of the silted pea gravel could not be determined because the silt washes out in the testing apparatus. The above data indicate that the gravel allows an inflow that uses the full capacity of the ground whereas the sand permits only a limited use of this flow capacity.

Filtering Property

The filtering effect of the pea gravel is certainly strange. At the end of a six-month period tests were made by washing the silt out of 100 ml of the wet dirty gravel through a 40 mesh screen and weighing the dried residue. Distilled water was used for the washing to avoid an increase in the weight of the total solids. The wet top silt contained 50.2 gr of dry fine material, the top layer of the gravel 16.3 gr, and the bottom layer 6.8 gr. An artificially prepared saturated mixture of pea gravel in which all pores were filled with silt, contained 20.0 grams of dry fine material. This shows that the top layer of pea gravel still had open pores. No holes could be seen in the silt cover. These data are averages of six samples from each layer.

From areas where the cleaner had been used, the gravel was free of a silt layer and its top one inch layer had only 2.4 grams of silt. This indicates the effectiveness of the cleaning operation.

A calculation of the Reynold's Number, as used for porous media,[6] $R = \frac{vd}{\nu}$ and taking $\nu = 10^{-5}$ gives:

For sand, 1 mgd on 7250 sq. ft.
$v = 18.4$ ft/day or 0.000,215 ft/sec.
$d = 1$ mm = 0.04 inch mean size
$R = 0.86$

For gravel, 3 mgd on 5700 sq. ft.
$v = 70$ ft/day = 0.000,81 ft/sec
$d = 4$ mm = 0.16 inch mean size
$R = 13.0$

As the transition from laminar to turbulent flow occurs at Reynold's Number 2 to 8, this shows that the flow through the sand was laminar whereas the flow through the pea gravel was turbulent. This may be the reason for the difference in action, but further studies are needed. Laboratory tests showed that the gravel acts only as a filter if the flow is less than 2 inches per minute. In the State Water Survey pit this velocity is 0.054 ft/min which is about one-third of the limit. However in the industrial plant pit, the velocity is 0.174 ft/min and, therefore, at the limit.

Emptying of Pit

Nearly every textbook of hydraulics teaches that the time of emptying a vessel through a small orifice is twice the retention time required for this same vessel when in equilibrium with the inflow.[7]

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6. Jacob, C. E. "Flow of Ground Water" in Rouse, H. Engineering Hydraulics, p. 322, John Wiley & Son, 1950.
7. Gibson, A. H. Hydraulics and Its Applications, p. 130, D. van Nostrand Company, 4th Edition 1930.

It can be shown that the same conditions exist when the vessel has uniformly porous walls. This fact was used in checking the wall conditions of the pit and of interest was that the factor 2 for the ratio was observed several times although the pit has no theoretically thin walls opening into free space. On other occasions it was found that the emptying time was from 5 to 30 times the retention time. In general the factor 2 was approached in high rates of flows whereas low flows gave higher factors.

Influence of Floods

Every time the river level rises above the pool stage the recharge rate increases in the pit. However the quantitative increase is not proportional to the rise in the river level. In fact, the increase is not even the same at different times with the identical river stages. Figure 5 gives some data on flow against river stage. These data were obtained under various conditions of silting.

It was shown previously[8] that a rise in the river stage causes a rise in the ground water level which disappears again with a fall in the river stage. On the other hand an increase in flow also increases the friction head loss in the piping system.

Considering these conditions the following data are surprising. At an inflow of 3 mgd the pit stage was 2.8 feet below the river pool stage. When the river rose 3.8 feet the inflow rose to 5.46 mgd. The pit stage was then 6.4 feet below river or 2.7 feet below pool stage, that is, only 0.1 foot above its stage at pool level. It seems improbable that this small rise of 0.1 feet in pit stage which caused only 0.5 per cent increase in wetted area could cause an increase in the flow rate of 82 per cent, especially when it is considered that the flow-away slope was also reduced. Yet such data have been found time and time again.

In this respect it is interesting that the relation of inflow to pit elevation is far from being lineal, nor proportional to the wet area or the area of the slope.

Recent data obtained under uniform slightly silted conditions show:

Inflow mgd	Pit Water Elevation ft. MSL	Wet Area sq. ft.	Slope Area sq. ft.
5.5	437.0	5980	3480
3.5	436.9	5950	3450
2.0	436.1	5450	2950
1.5	435.4	5000	2500
1.1	434.9	4700	2200

These data are shown in Fig. 6. Plotting the pit elevations against inflow yields a curve whose characteristics indicate a limiting water elevation regardless of inflow. This limit raises with increased silting in the pit. While the difference between the river stage and the pit water level is governed by the supply line, the capacity of the pit to infilter the higher rates with a small increase in area is independent of the supply line and cannot be explained as yet.

8. "Apparent Changes in Water Storage During Floods at Peoria, Illinois" by Max Suter. Transactions. American Geophysical Union, Volume 28, Number 3, June 1947.

Effect of Recharge

The effect of the recharge has been considerable. In the earlier seasons, when the inflow was around 1 mgd, the rise per season in ground water level near the pit was from 3 to 4 feet, a quarter mile away about 1 foot, and at a distance of a half mile a few inches. With 3 mgd inflow the rise near the pit was up to 7 feet, a quarter mile away about 5 feet, and a half mile away up to 3 feet.

The great amount of inflow of cold river water, much of it with near freezing temperature, caused a cooling effect on the ground water. Temperature measurements in the wells show that the recharge water floats on top of the existing ground water even in the case when the recharge water is much cooler. Within a well, temperature differences up to 20°F have been observed. The differences in the specific gravity of water due to temperature seems to be insufficient to overcome the internal friction and produce a turnover as it occurs in lakes.

On the other hand pumping by wells, even if they have a screen only in the lower part of the aquifer, pulls in a great amount of the upper cold layer. In an industrial plant, pumping wells produced water of a temperature as low as 42°F whereas previously the temperature in these wells varied only from 52°F to 56°F.

Chemical Tests

Chemical and physical tests are made on the river water and at a 58 foot deep well at the laboratory, only 190 feet from the pit. These tests are made 4 times a week the year around.

Physical tests consist of temperature and turbidity measurements. The chemical tests show great variations in composition of the river water.

	Range in ppm	Average
Hardness	196 - 385	285.9
Alkalinity	102 - 240	203.6
Sulfates	64 - 186	118.3
Chlorides	15 - 45	25.8
Dissolved Oxygen	6.0 - 14.0	10.9

Similar variations exist in the well water.

	Range in ppm	Average
Hardness	208 - 352	292.4
Alkalinity	156 - 340	206.8
Sulfates	79 - 137	117.5
Chlorides	19 - 46	29.5
Dissolved Oxygen	0.1 - 11.8	5.3
Iron	trace - 0.2	0.1

The first chemical indication of an inflow from the pit to the well is in the appearance of dissolved oxygen and the disappearance of iron.

Residual chlorine is found in the well water only in occasional traces. The highest residual chlorine appeared after two of the shutdowns of the pit while the ground water mound was lowering.

The other chemical properties are in the same quantitative range and cannot furnish a measure of the rate of infiltration. In general their changing trends are parallel more during the infiltration period than during the summer although great irregularities to this rule occur at times.

Chlorination

The river water is chlorinated as a sanitary safety measure. For two seasons the average chlorine dose was 8.8 ppm. This produced a water of satisfactory quality at the laboratory well. From the economic standpoint the least amount of chlorine should be used which will produce an acceptable quality. Using various rates of chlorine it was found that 3 ppm will do this. When only 2 ppm were used the number of bacteria in the well almost tripled. When no chlorine was used in a 3 day test the bacteria count increased considerably. A 10 ppm chlorine dose was used after this test to sterilize the soil and ground water.

Bacteriological Tests

Bacteriological tests are made on the river and well waters 4 times per week the year around. The tests on river and well water consist of a total count on nutrient agar. In addition, the lactose fermentation tube test is made on the well water with attempts for confirmation, using E.M.B. agar and brilliant green bile media.

The total counts in the river water vary between 12,000 and 66,000 bacteria per milliliter with an average of 26,000.

The total counts on the well water vary between 5 and 20 bacteria per milliliter with an average of 10, exclusive of results obtained during test periods with low chlorine treatment.

A relatively high number, about one per cent, of positive lactose fermentation tubes are found. However confirmation tests have always been negative with the use of pea gravel and 3.6 ppm chlorine whereas when sand and 8.8 ppm chlorine was used some confirmation for coliform organisms was obtained.

Algae

Several times the operation of the pit was slowed down by the sudden appearance of growths of algae. At times these organisms formed a green layer in the pit and the chlorine was not an effective control. Copper citrate and copper sulfate were used to kill the growth with the copper citrate having a somewhat longer lasting effect but costing more. The heavy influx of algae never lasted over 5 days and stopped as suddenly as it appeared. Every attack consisted of many types of algae with a different type predominating each time. Their appearance was very irregular and unpredictable as to season or temperature of the water.

SUMMARY

In Peoria success was obtained in designing and operating an efficient recharge pit capable of recharge rates from 23 to 27 mgd per acre exclusive of flood times when these rates are exceeded.

The operation of this pit is for research purposes to determine the factors affecting the artificial recharge of ground water reserves. However attempts to explain the various findings have so far met with little success. Many unexpected facts have been observed which simply are not as the books say they should be. Recharge operations still offer a wide field for research. It is hoped that funds will continue to be available for this research and that a reliable theory can be developed for flow from recharge pits which can be used in the design of efficient pits of high rate of inflow.

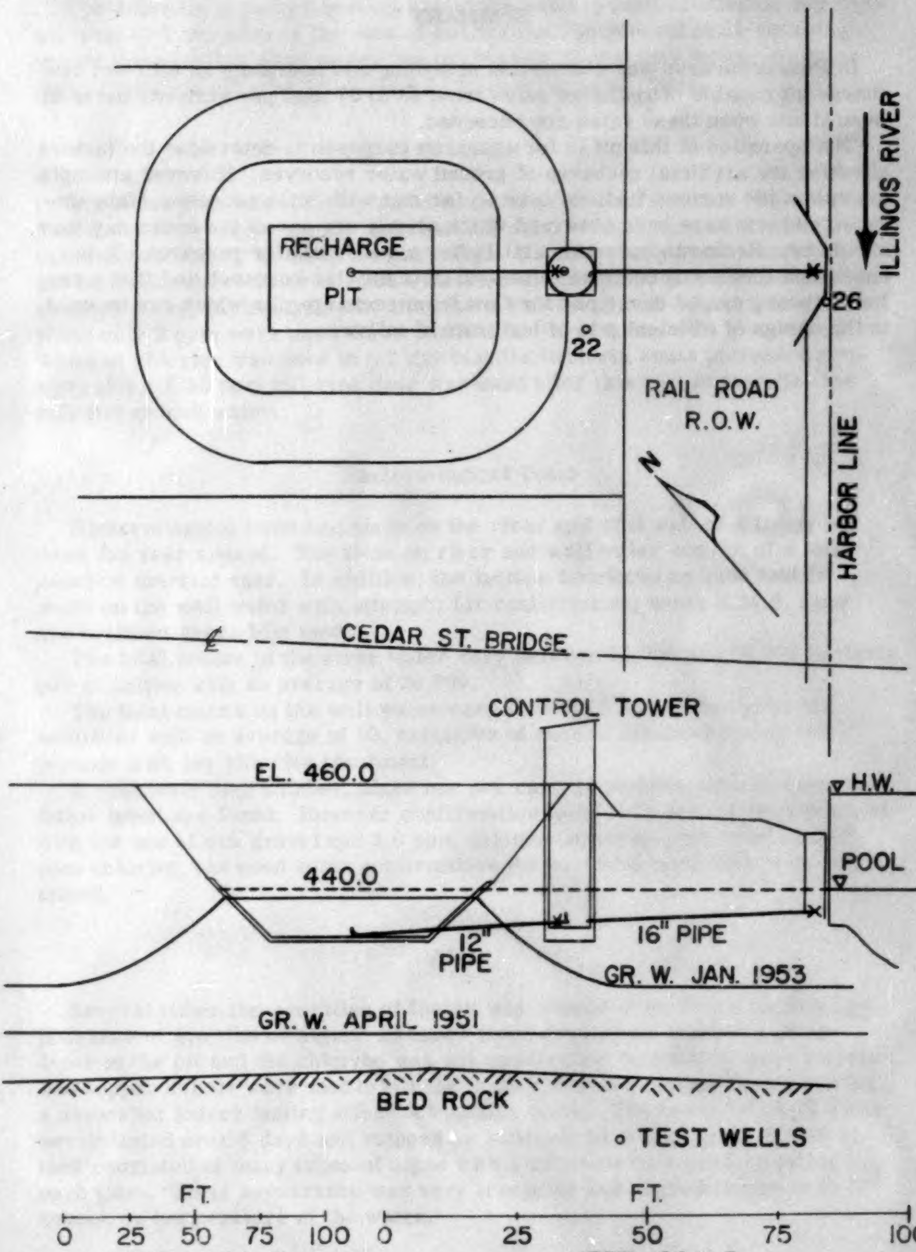
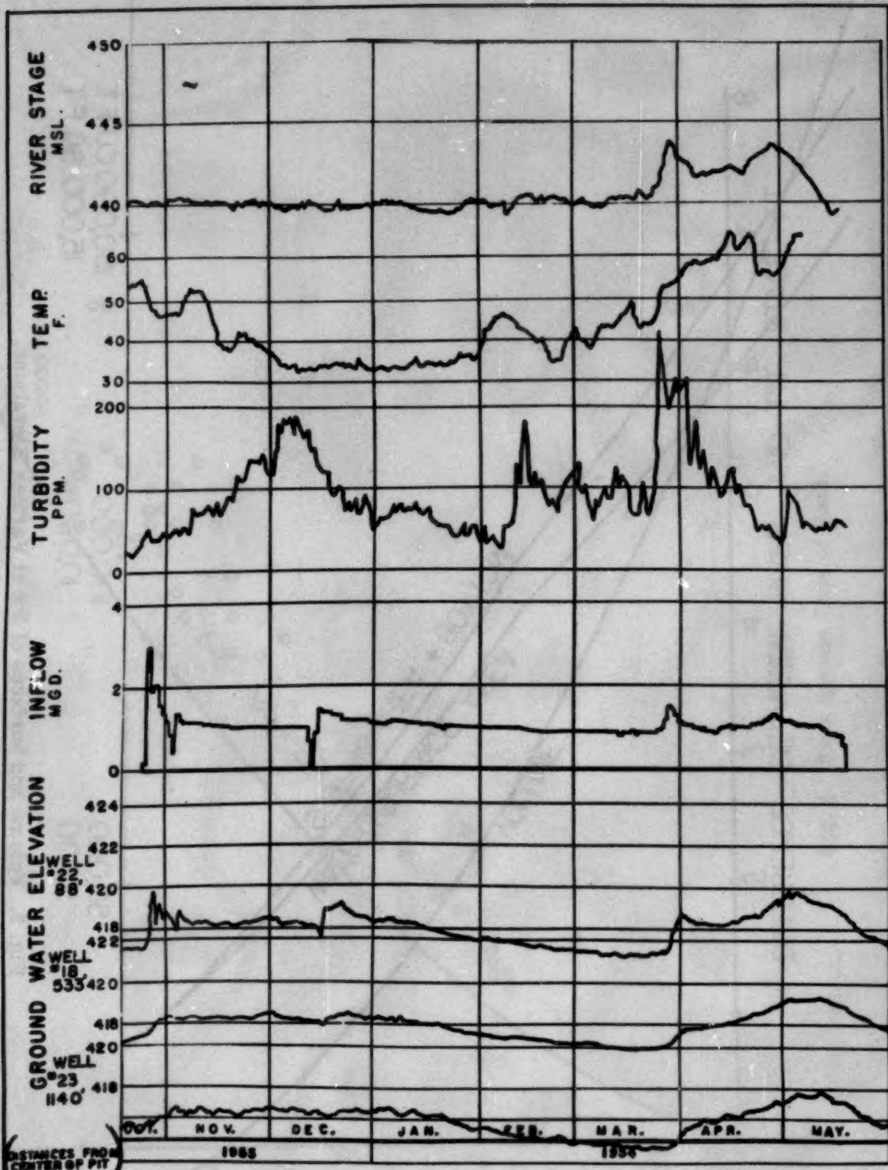


Fig. 1. Plan of Layout and Longitudinal Section of Recharge Pit.



CHARACTERISTIC TIME CURVES PEORIA, ILL.

ILL. STATE WATER SURVEY DIVISION

Fig. 2. Summary of Seasonal Inflow 1953-54.

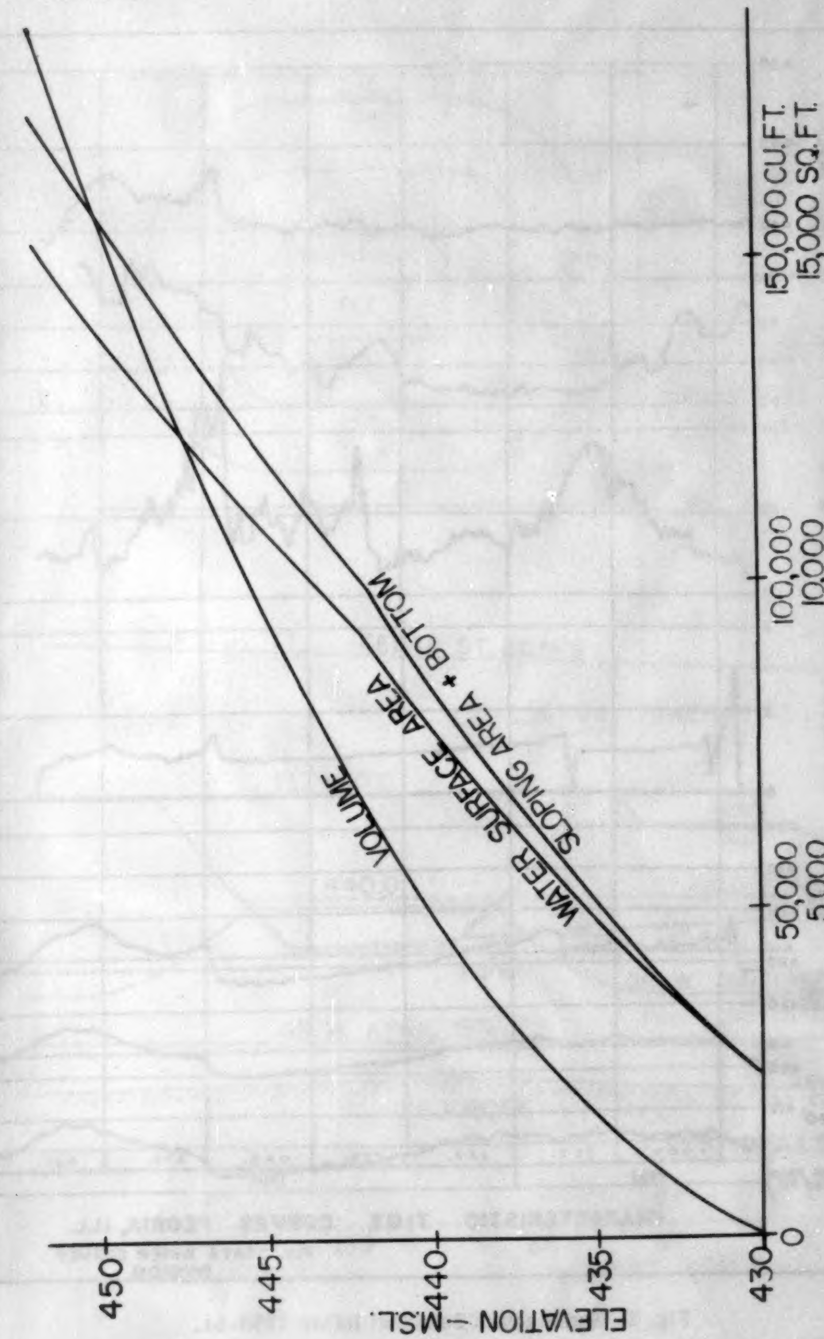


Fig. 3. Volume and Surfaces of Pit at Various Elevations.

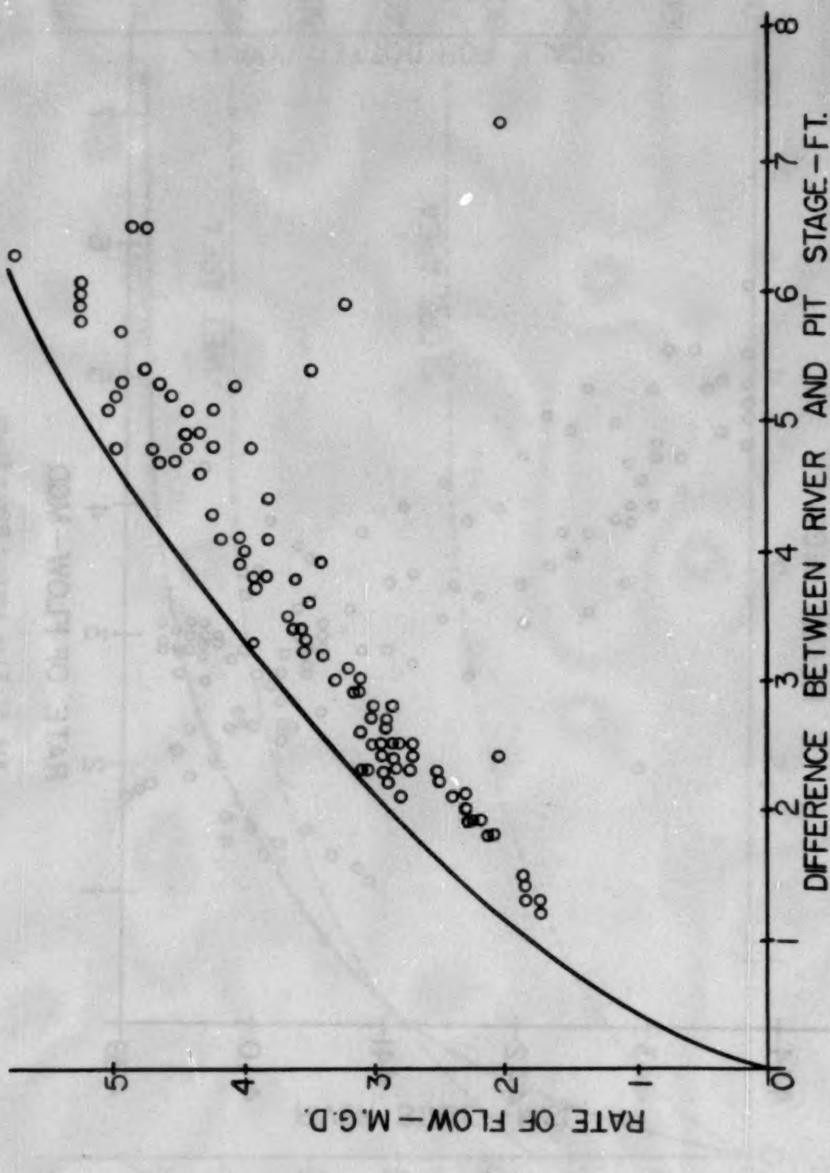


Fig. 4. Flow against Loss of Head.

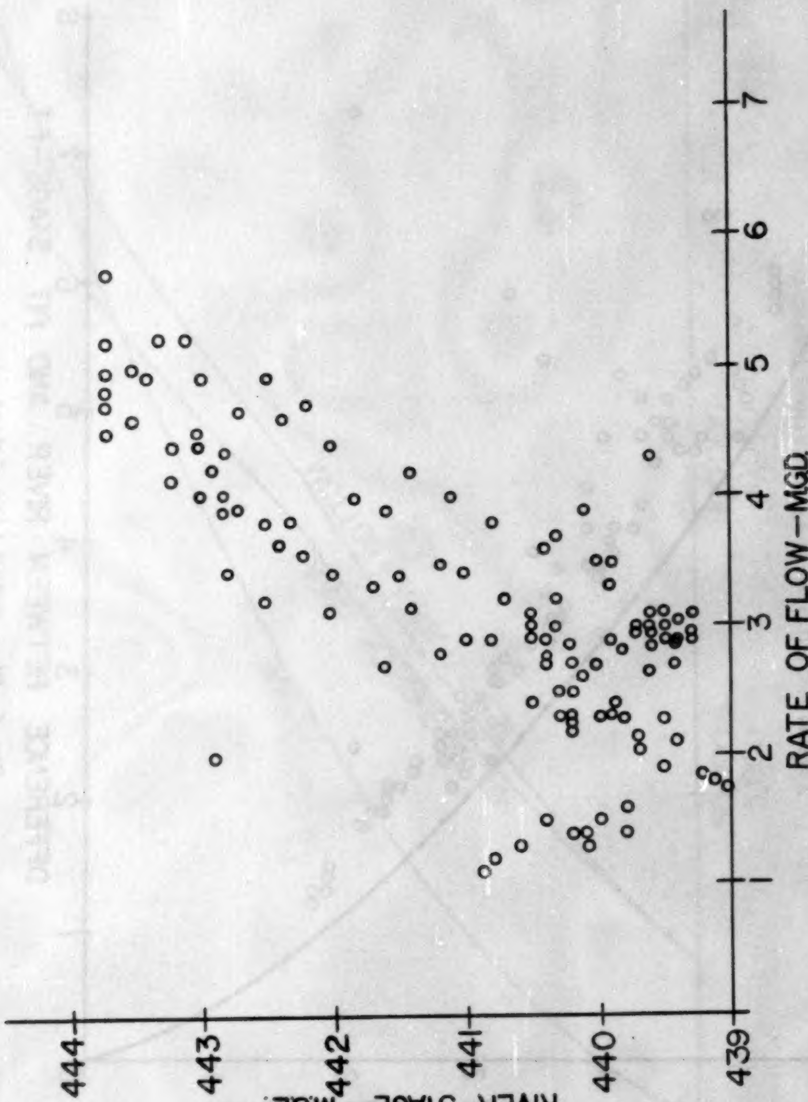


Fig. 5. Flow against River Stage.

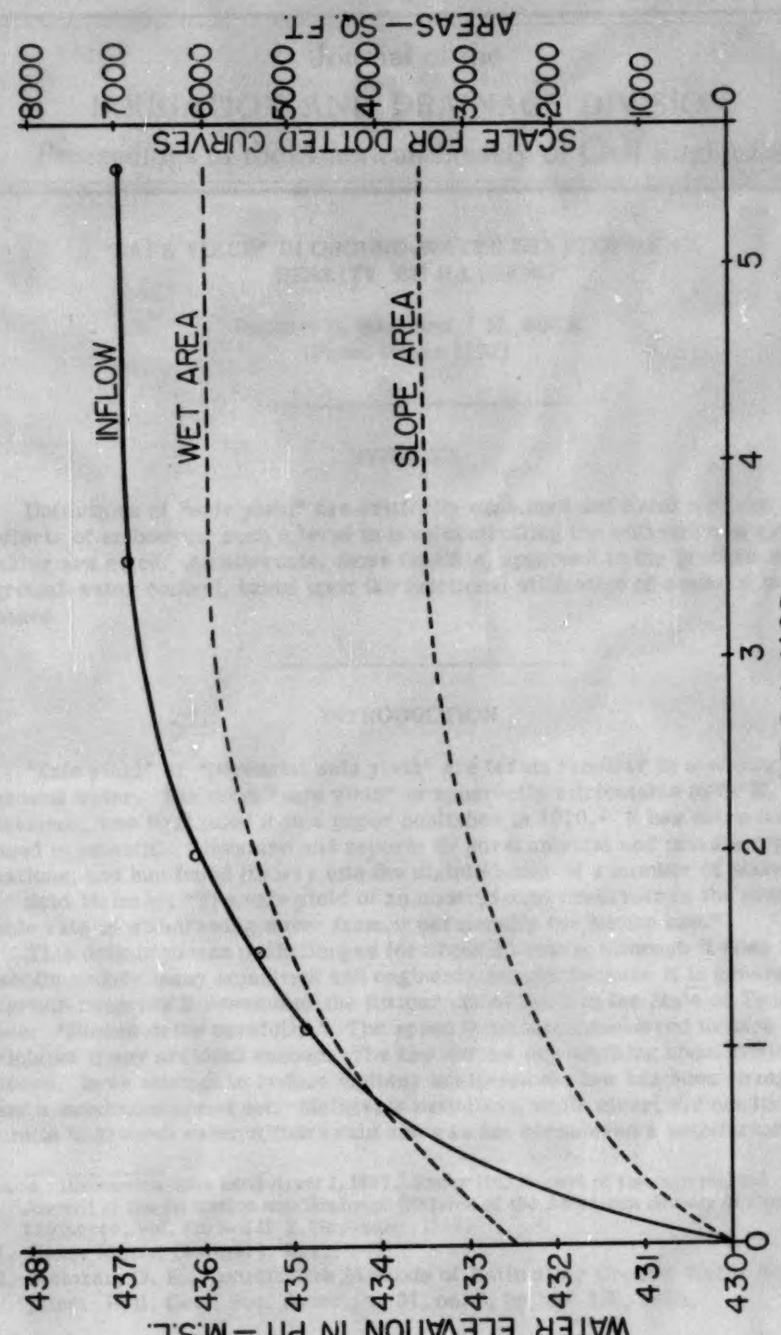


Fig. 6. Flow against Pit Water Elevation.

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"SAFE YIELD" IN GROUND-WATER DEVELOPMENT, REALITY OR ILLUSION?

Raphael G. Kazmann,[1] M. ASCE
(Proc. Paper 1103)

SYNOPSIS

Definitions of "safe yield" are critically examined and found wanting. The effects of embodying such a term in laws controlling the utilization of ground water are noted. An alternate, more feasible, approach to the problem of ground-water control, based upon the functional utilization of aquifers is proposed.

INTRODUCTION

"Safe yield" or "perennial safe yield" are terms familiar to students of ground water. The term "safe yield" is apparently attributable to O. E. Meinzer, who first used it in a paper published in 1920.[2] It has since been used in scientific literature and reports by governmental and private organizations, and has found its way into the statute books of a number of states.

Said Meinzer, "The safe yield of an underground reservoir is the practicable rate of withdrawing water from it perennially for human use."

This definition was unchallenged for about 20 years, although it does not wholly satisfy many scientists and engineers simply because it is general. In certain respects it resembles the former speed limit in the State of Tennessee: "Please drive carefully." The speed limit was considered to have been violated if any accident ensued. The law did not say anything about limiting speed. In an attempt to reduce highway accidents the law has been changed and a maximum speed set. Meinzer's definition, while clear, did not fix limits to ground-water withdrawals and was not considered a satisfactory

Note: Discussion open until April 1, 1957. Paper 1103 is part of the copyrighted Journal of the Irrigation and Drainage Division of the American Society of Civil Engineers, Vol. 82, No. IR 3, November, 1956.

1. Cons. Engr., Stuttgart, Ark.
2. Meinzer, O. E., Quantitative Methods of Estimating Ground-Water Supplies: Bull. Geol. Soc. Amer., v. 31, no. 2, pp 329-338, 1920.

guide in deciding whether pumpage had exceeded the "safe yield."

As pumping rates increased and the water levels in ground-water basins declined, Meinzer's definition was modified and made more specific. The words "safe yield" came to mean an upper limiting pumping rate, the limit being set, generally and ideally, by the pre-pumping steady-state flow in the aquifer (assuming that such flow did occur). Attention and thought was then directed to the phenomenon of ground-water replenishment, the so-called "recharge" of aquifers.

The thought became current that the "safe yield" of an aquifer was surely determinable in advance of ground-water development—or even after development had begun.

By 1949 Meinzer's definition had been carefully reconsidered and "safe yield" was redefined by a synonym "perennial yield": "The perennial yield of the aquifer underlying an area of ground-water development has been regarded as the maximum rate at which water can be salvaged from the natural discharge, or added to the recharge, or both . . . In some reports the economical pumping lift has been considered to be a factor in this definition, however, the economics of recovery seem to be irrelevant to the determination of the quantity of water which an aquifer will yield and so are not considered here."[3]

The Federal Government, in 1951, issued a circular on the water situation in the United States, with special reference to ground water.[4] In a special, state-by-state summary, under a heading "Deficiencies in Information" there is listed time and time again, "Safe yield of basin not known at present" or a paraphrase of this statement.

Evidently the Federal Government has decided that for each ground water basin there is a single, determinable figure which is the "safe yield" of the basin and with only a reasonable extension of present efforts this "deficiency in information" can be alleviated. Such an attitude has already had its effect on the laws of a number of states and will undoubtedly influence the course of future legislation in the field of water resource development.

However, in view of the fact that far more water is stored in waterbearing formations than in all man-made reservoirs, it is imperative that basic technical concepts, and the legal consequences of these concepts, be examined carefully and critically.

The Heart of the Matter

The basic technical question is this: "For any given waterbearing formation, is it theoretically possible to determine a single specific figure which can be termed the "safe yield" of the aquifer, without reference to the total water resources of the region and adjacent regions?"

As point of departure, let us examine a definition of "safe yield" used by the Federal agency which has publicised and still uses the term, in view of the fact that it is a scientific agency and is not principally concerned with

3. Williams, C. C. and Lohman, S. W., Geology and Ground-Water Resources of a Part of South Central Kansas: Geol. Surv. Kansas, Bull. 79, 1949, p 212.
4. McGuinness, C. L., The Water Situation in the United States with Special Reference to Ground Water: Geol. Surv. (U. S. Dept. of Interior): Circ. 114, June 1951.

engineering problems, construction methods, or the costs of operation of structures. The last definition quoted will serve. It can be deduced that economics, as shown by the definition, should be excluded from any definition of "safe yield."

Therefore, as a first approximation: "safe yield" is a term apparently used to designate the quantity of water flowing through an aquifer before water was first pumped from it. It can be taken for granted that if the pumping rate from the aquifer does not exceed this figure such a rate of withdrawal may be continued indefinitely. But can it?

Water may be withdrawn from an aquifer by means of a pump placed in a suitable structure such as a well, infiltration gallery, or horizontal collector. Is it, therefore, correct to state that if the structure (and for this purpose any fixed number of wells may be considered one "structure") is designed hydraulically to produce a quantity of water equal to the "safe yield" of the aquifer and provided with suitable pumping equipment, the "safe yield" will not be exceeded and the structure may be expected to continue to produce water at this rate indefinitely?

There are a number of sound reasons why such a pumping rate might not be continued indefinitely, even though it might be possible for an initial period. Let us consider two of these:

1. The transmissibility of the formation, combined with the maximum available hydraulic gradient, might become too small to permit the necessary rate of flow through the formation to the producing structure (or structures).

2. Even though enough water might flow to the structure at all times, the quality of water throughout an aquifer is not necessarily the same. Concentration of the withdrawal of water from a limited area might cause underlying mineralized water to enter the structure, modifying water quality to the point of being unusable—even though the "safe yield" of the aquifer were the fresh water "safe yield."

In these two cases a perennial water output equal to the "safe yield" might not be reached.

Now consider a situation wherein the natural recharge of the aquifer occurs by seepage from a stream traversing the outcrop area of the aquifer. The producing structures, designed for the "safe yield" of the aquifer (which might be computed on the basis of the average transmissibility of the aquifer and the average position and gradient of the piezometric surface before the start of pumping) are located on the river bank next to the area of recharge. In pumping an amount equal to the perennial "safe yield" of the aquifer, the short distance between well and recharge area might require use of only a small fraction of the available drawdown. Consequently, by increasing the drawdown a steady-state output might be obtained in excess of the "safe yield" of the aquifer, depending only upon the availability of water in the stream to replensih the aquifer at the rate at which water is withdrawn. The Des Moines infiltration gallery illustrates this situation admirably, having been constructed in a thin aquifer next to a river and producing far more than the original flow through the aquifer before the start of pumping.

It appears that our approximate definition cannot be used: for a single aquifer it might give a value of "safe" water withdrawal that is simultaneously too high and too low! The crux of the trouble seems to be that the engineering practicability of accomplishing a rate of water withdrawal equal to the "safe

yield" has been excluded—and the practicability, which means economics, was excluded on purpose!

The definition of "safe yield" should probably be modified: it is not mere "observed data" but it requires that economics and engineering be considered. It may be legitimate, at this point, to speculate as to what procedure the Federal Government has devised to produce a single "safe yield" figure for each aquifer or "ground-water basin" from its purely scientific and fact-finding studies since, in 1955, with reference to ground water it has been proposed, "That for the purpose of gaging water supplies and estimating the perennial yields of underground basins a sufficient number of Federal observation wells be installed."[5]

It seems evident that the determination of "safe yield" is far different from measuring how much water flows in streams or what is the depth to water in various parts of an aquifer.

The purely scientific approach of Meinzer's successors having failed, the engineering approach remains:

"Safe yield is the annual extraction from a ground-water unit which will not or does not (1) exceed the average annual recharge; (2) so lower the water table that permissible cost of pumping is exceeded; or (3) so lower the water table as to permit intrusion of water of undesirable quality."[6] A paraphrase of this definition appeared in WATER, the Agriculture Yearbook for 1955.[7]

This definition includes "average annual recharge" and qualifies it by excluding highly mineralized water and making allowance for the "permissible cost of pumping." Although this definition is highly useful in another connection, it leaves much to be desired as a usable definition of "safe yield."

For instance, a law based on achieving "safe yield" as defined above might be slightly difficult to enforce: an administrator would have to decide upon permissible pumping cost—and this cost might be much higher for an industry or municipality than for a neighboring farmer irrigating wheat or cotton who was pumping from the same aquifer. In addition, the "average annual recharge" must be determined, and this is a term possessing all of the weaknesses of the previous definition of "safe yield."

It may be legitimate to speculate how a State Engineer can proceed to find a basis for controlling ground-water withdrawals using perennial "safe yield" as his criterion, when he can't define it.

Yet, these definitions should not be dismissed lightly. They are the work of thoughtful men, eminent in geology and engineering, who had much experience in ground water. The difficulties with the definitions are to be considered a warning that it is necessary to stop redefining "safe yield" and reexamine the entire situation. But before doing this it is desirable to discuss the effect of the "safe yield" doctrine on water-resource development.

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5. Commission on Organization of the Executive Branch of the Government, Task Force Report on Water Resources and Power: Vol. 3, p 1077, June, 1955.
6. Conkling, Harold, Utilization of Ground Water Storage in Stream System Development: Trans. Am. Soc. Civ. Eng., Vol. III, pp 275-302, 1946.
7. Muckel, Dean C., Pumping Ground Water so as to Avoid Overdraft: "Water," Yearbook of the Dept. of Agriculture, pp 294-5, 1955.

"Safe Yield" and the Doctrine of Appropriation

In the Western States, where water is limited in quantity and is particularly valuable, more thought has been given to public policy in the field of water control and utilization than anywhere else on earth. It may be fairly stated that the doctrine of appropriation is becoming dominant in the West with respect to the development of surface water. Efforts have been made to extend this doctrine to cover utilization of ground water.

According to the Supreme Court,

"To appropriate water means to take and divert a specific quantity of water therefrom (the watercourse—author's note) and to put it to beneficial use in accordance with the laws of the state where such water is found, and by so doing to acquire a right under such laws, a vested right to take and divert from the same source and to use and consume the same quantity of water annually forever."[8]

As a rule, the first person to put water to beneficial use enjoys a superior right, a priority, over any later, "junior," appropriators. Inherent in the Supreme Court's definition is the "perpetual right" to divert water conferred by the state upon the property owner. This implies that a determinable, economic quantity of water will always be available, even though the total quantity available in any year may differ greatly from the quantity available during another year.

Appropriators pumping directly from a stream have little difficulty in apportioning the water according to the priority doctrine. When all available water is required by senior appropriators, the junior appropriators either have no water to pump or are prevented from pumping by a water master. As more and more of the mean annual flow in a stream is appropriated, junior developments become increasingly risky and expensive due to the increasing possibility of economic failure of such developments during periods of prolonged sub-normal stream flow.

Appropriation doctrine could be applied directly to ground-water withdrawals if—if the source of ground water were completely independent of precipitation, evaporation, and runoff. Under such circumstances it might be possible to determine the annual contribution of the ground-water basin and this would be in addition to all flows measured elsewhere. Hydrologic fact contradicts the postulated condition:

A particle of water can be part of the surface-water body, infiltrate into an aquifer, be pumped out for irrigation or sanitary purposes, reach a stream as return flow or sewage, and repeat the cycle several times before either being evaporated or reaching the sea.

Consumptive use of surface-water that prevents it from reaching the outcrop of an aquifer and becoming part of the ground-water supply would interfere with the appropriative rights of ground-water users. Conversely, the increased recharge to an aquifer due to the operation of wells (next to a stream, for example) might adversely affect the rights of appropriators of surface water by diminishing the stream flow.

Thus, the concept of a "safe yield" of aquifers, independent of considerations of regional hydrology, cannot be reconciled with the doctrine of appropriation. All water pumped from the ground must be replaced by water

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8. Arizona vs. California, 238 U. S. 423 (1931).

coming from the land surface if a perennial water supply is to be obtained from the ground. If all surface runoff in the area overlying an aquifer has been appropriated, a perennial supply cannot be obtained from the ground, laws notwithstanding.

Nevertheless the built-in difficulties found in "safe-yield" legislation do not come to light immediately after the passage of the law. For example, only within the past five years has the New Mexico ground-water law, passed in 1931, been challenged for the reason that a ground-water basin cannot be administered on the basis of appropriation or the impariment of existing rights.[9] Additional litigation along the same lines may be expected in the future.

The Role of Aquifers

How, then, is the problem of ground-water conservation and development to be solved within the framework of the present development of surface water resources?

Consideration should first be directed to the composition and structure of waterbearing formations, their functions, and the limitations of each function.

An aquifer is a volume of the earth's mantle containing water. This volume of material is sufficiently permeable that water can move through it under the action of gravity and from it water can be withdrawn at useful rates by properly designed structures. An aquifer composed of unconsolidated material such as sand, or sand and gravel, functions as a natural "filter plant," a "pipe line," and a "reservoir." In certain rock formations, such as cavernous limestone or highly fractured basalt, the filter plant function may be partially or entirely lacking. The discussion of aquifers as filter plants does not apply to such rock aquifers.

Aquifers as Filter Plants

The exposed area of an aquifer which is composed of granular materials acts as a natural filter to remove all suspended matter and pathogenic organisms from the surface water which passes through it into the ground. Sometimes this area becomes clogged and recharge is rejected. Sometimes the aquifer is brimful of water and recharge is also rejected. When, for one reason or another, conditions favorable to recharge occur, the aquifer resumes its function as a natural filtration plant.

It is by virtue of this property of aquifers that ground water reaching wells is free of suspended matter and pathogenic organisms. This phenomenon of safe water from an aquifer composed of granular materials has been observed even when the aquifer is traversed and recharged by highly polluted streams, as in the industrial East. Even organic tastes and odors are lacking in the ground water, even though the source may contain them in full measure. It has been suggested that an act of bio-filtration occurs in the recharge area of the aquifer: that the aquifer functions not only as a rapid sand filter, but also as a bio-filtration plant.

The capacity of an aquifer to act as a filter plant is not constant throughout

9. Thomas, H. E., Water Rights in Areas of Ground-Water Mining: Geol. Surv. (U. S. Dept. of Interior) Circ. 347, Washington, 1955, p. 10.

the year. It depends on the head, temperature, velocity, and silt content of the water, the design, location, construction and operation of the water-producing structures, and other, less important factors. However, the probable minimum yield of a well field constructed in or near a recharge area can be computed to the required degree of accuracy after the proper engineering exploration and hydrologic studies have been accomplished. The economics of water development play an important part in determining the "firm" yield of the system.

It has been pointed out elsewhere, that in the East, where streams are highly polluted, the filter-plant function of aquifers is of great economic importance.[10] The use of water for sanitary and industrial purposes places a value on clean water, even though there is no shortage of sediment laden, polluted surface water. In the West the emphasis is primarily on the availability of water. The presence of sediment and microorganisms in water are of minor economic importance when the water is used for irrigation.

Let it be noted that since the yield of the "filter plant" is importantly changed by rainfall, runoff, and water temperature, its maximum utilization depends on the magnitude of available storage to permit this variable output to be averaged over a period of time.

Aquifers as Pipe Lines

Although the use of aquifers as pipe lines (to convey water from the recharge area to the well field) has been of great economic importance in the past, it is likely that in the future the importance of this function will decrease. A pipe line full of sand is not an efficient vehicle for transporting water over long distances, even though the pipe line may be an aquifer extending over many hundreds of square miles and may be hundreds of feet in thickness. This is particularly true in areas of intensive ground-water development, where use as a pipe line may well conflict with other uses of the aquifer.

For example, under a gradient of 20 feet per mile, an 18-inch concrete pipe will transmit about 5 million gallons of water a day. Under the same gradient it requires an aquifer one mile wide, 75 feet thick, with a permeability of 3,400 gpd/sq ft to move the same quantity. Aquifers are inefficient pipe lines for the long distance transport of water, although, for distributing water over relatively short distances there is much to be said in their favor. The writer's experience would seem to indicate that the maximum distance that an aquifer should serve as a pipe line is about two miles. In areas of intensive ground-water pumpage this maximum may be too high.

The foregoing discussion contains important implications regarding the need for ground-water recharge in or near the area of pumpage: from an engineering viewpoint artificial recharge is necessary whenever a well field is located at a great distance from the area of natural recharge. Where direct recharge from the land surface is impossible, due to the existence of clay layers between the surface and the aquifer (as in the Grand Prairie Region of Arkansas and many parts of the Central Valley of California), artificial recharge through man-made structures is essential.

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10. Kazmann, R. G., The Role of Aquifers in Water Supply: Trans. Am. Geophys. Union, v. 32, pp 227-230, Washington, 1951.

Aquifers as Reservoirs

Many ground-water problems originate because of erroneous conclusions reached as a result of short-period observation of the operation of wells in depleting, or failing to deplete substantially, the enormous quantity of water stored in aquifers. Many aquifers are essentially vast, closed reservoirs which are analogous, on a large scale, to petroleum reservoirs. There is no merit in discussion the "perennial yield" or "safe yield" of such an aquifer. No one uses the term relative to an oil-bearing formation, yet many reports have been written (by the writer, among others) purporting to determine the "safe yield" of closed aquifer's. In oil fields the rate of withdrawal and well spacing are subject to regulation because these factors determine to a large extent the maximum total output from the reservoir. But no one acts or thinks as though a perennial supply or oil were available, no matter how vast the oil-bearing structure. We all know and accept the fact that a limited, though large, quantity of oil may be obtained from a given area. This fact is even recognized in the Federal tax laws by the much-debated "depletion allowance."

It is probably due to the limited popular understanding of the tremendous quantities of water stored in our major aquifers, combined with a misapprehension as to how, and how fast, ground-water replenishment occurs, that the renewability of our water resources, as evidenced by the perennial flow of streams, has been embodied in the ground-water laws of several states. These laws try to limit the pumpage of ground water to the "safe yield" of ground-water basins. But unless surface water is available to replace the ground water that has been pumped and is used for that purpose, it is unsound public policy to discuss the "safe yield" of a water-bearing formation.

The preceding statements should not be interpreted to mean that ground-water reservoirs are of no importance in connection with the conservation and use of our water resources. Such reservoirs are essential in the conservation of our water resources. They make possible the utilization of flood waters far beyond the potentialities of surface reservoir sites. The water stored in them is not subject to evaporation loss and the reservoir area is not removed from productive use. Moreover, most of our major aquifers underlie areas of human habitation and agricultural activity such as upland plains, river valleys, and coastal plains. The storage of surplus surface water under or near the areas of water use has manifest advantages (two of which have been pointed out). The principal drawbacks are, (1) the impossibility of generating electric power from water stored beneath the surface of the ground and (2) the impossibility of using the stored water directly for recreational purposes. However, as a counterbalancing factor, the increasing use of atomic energy in modern civilization has made the storage of water in aquifers of increased importance: such stored water is not subject to direct contamination by radioactive fallout and, by virtue of dilution and the slow movement of water within aquifers, even if the recharge-water is contaminated, radioactive decay has time to occur and an important amount of dilution also occurs before the water is pumped out and used. As a rule, water stored in aquifers is safe water.

In utilizing aquifers as storage reservoirs, it should be emphasized that a reservoir must be filled as well as emptied—and if the replenishment is negligible or non-existent, no perennial right to withdraw water should be granted since the available water supply is not perennial.

It is becoming evident that the technical breakthrough most needed at this time in the field of ground water is a generally applicable method of artificial recharge which will use raw surface water that is either untreated or has received an absolute minimum of treatment. Without such a breakthrough we cannot properly utilize our aquifers as ground-water reservoirs.

Artificial Recharge

An amusing summary of the present status of recharge was given at the "Water for Texas" conference in 1955. It was reported on page 26 of the Water Log, November, 1955:

"There were also discussions on recharge, the least known phase of ground-water development. It seems that the best method of recharge is accidental recharge. The few attempts at artificial recharge have met with many problems such as silting up of the wells, insufficient pressure, limited transmissibility of sand strata, organisms that multiply in air and clog wells, and many others."

What are the prospects of achieving a technical breakthrough in the field of artificial recharge? The prospects are good. The problem is one of engineering, not science. Engineering development is needed to reduce the cost of artificial recharge to a point far below its present level. For instance, the use of completely filtered river water has been entirely successful in recharging depleted aquifers through wells and horizontal collectors. Artificial recharge during and after the war years was successfully accomplished with the use of filtered City water in Louisville and Cincinnati. However, the cost of the water was high. More recent experiments in the Los Angeles area, utilizing the treated effluent from a sewage plant, have been entirely successful. The mathematics and hydraulic theory have been found to be adequate: it is now a question of engineering technique and cost. Estimates have been made which seem to indicate that a properly designed development project in the field of artificial recharge, to utilize surface water which had a minimum of treatment, or no treatment at all, would cost between $500,000 and $1,000,000 for the needed exhaustive tests on structure prototypes. There is every reason to believe that an economic method of artificial recharge through structures will be achieved within the next few years. Such a development would enable the nation to fully utilize aquifers as storage reservoirs and make possible great advances in the field of water conservation.

Ground-Water Legislation

Legislation provides the framework for the development of our water resources. It must be broad enough to include present and potential technical developments.

In most of the West the doctrine of appropriation has been adopted for the development of our surface-water resources. The writer has no doubt that this doctrine can be successfully extended to include ground water, although its equitable application will involve complex matters of geology and engineering, far removed from the relatively simple matter of flow measurement and control of water in open channels or closed conduits. The doctrine will be more easily applied in areas where unappropriated surface water is

available and in areas where "safe yield" or an equivalent phrase on ground water use is not presently incorporated in the law.

On the assumption that the water law in a State is based on the doctrine of appropriation, what are the elements that must be included in water control legislation applicable to ground water?

First, legislation should be passed defining the recharge of aquifers as a beneficial use of water and authorizing the creation of water districts for the purpose of ground-water recharge. These Districts would be formed on the initiative of local users of ground water and the District would have the right to appropriate water (or buy it) for the purpose of recharge. Organization of the District would be similar to the organization of an irrigation or drainage district. The District would have the necessary power to collect operating funds and to insure that the average annual extraction from the aquifer or aquifers within the District did not (1) exceed the average annual recharge, (2) so lower the piezometric surface that the permissible cost of pumping was exceeded, and (3) so lower the piezometric surface as to permit the intrusion of water of undesirable quality. Despite minor modification the reader will recognize the engineering definition quoted previously with respect to "safe yield."

Secondly, existing water law would be extended to ground-water withdrawals. All areas of ground-water pumpage would be classified in one of two categories: areas of "mining" or areas of "perennial yield." The distinguishing characteristic would be the availability and utilization of surface water for accidental or artificial recharge.

In areas of ground-water mining, well drilling and pumping would be unrestricted. However, some attention might be given to the desirability of establishing a minimum well spacing in order to insure that water could be recovered in economic quantities for the maximum period of time. All areas of ground-water development might automatically be classed as mining areas unless it could be shown that unappropriated surface water was available for recharge, in quantity sufficient to balance the withdrawals, and the aquifer was currently being replenished within a predetermined distance of the water-producing structures.

Areas of "mining" which had been organized as Districts would become "perennial yield" areas and would appropriate surface water on an equal basis with other surface-water users and would be obligated to construct, maintain, and utilize structures for artificial recharge. Within the District, well owners would receive secondary appropriative rights: these rights would be contingent upon the continued recharge of surface water to the aquifer. There would be no senior or junior appropriators within a District: the stored water within the aquifer would make it possible to put all appropriators on the same level of priority. Within the District there would probably be no restriction upon the drilling of wells, although the total quantity of water pumped during a period of years by an appropriator might be limited.

Within the boundaries of the District, outside of the area of "perennial yield," there would be established a "buffer zone" to minimize the possibility of capture of any substantial quantity of recharge water by wells whose output was not replaced by surface water obtained through the District's appropriative right. The area of the buffer zone might be fixed by administrative authority. The buffer zone is made necessary by the fact that it would be possible to establish a "perennial yield" area in a portion of an extensive aquifer from which, in another area, it might be desirable to mine water. It

should be mentioned that there are several technical devices available to make the buffer zone effective in preventing loss of ground water to wells outside the area of "perennial yield."

Before concluding this brief discussion of proposed legislation, legislation which might replace present ground-water law which requires the pumpage from an aquifer to be limited to the "safe yield," it is necessary to make brief mention of the "watercourse problem." The "watercourse problem" was aptly defined by Harold Thomas in 1951 as the ". . . . result from pumping wells along rivers, where the ground water is so closely related to the water in the stream that pumping from wells depletes the stream flow. Diversions from the stream for various purposes may increase the amount of ground water at one place and reduce it at another."[11]

It will be necessary to classify such areas of "accidental recharge" as perennial yield areas. Well owners in such areas would be required to organize themselves into Districts in order to receive an appropriative right on an equal basis with other diverters of surface water—or would have to stop operating their wells. However, since recharge would be "accidental" it will be necessary for such Districts to equalize stream flow above and below the area of accidental recharge in accordance with their appropriative rights. Such Districts might also have to undertake operations of artificial recharge. It is likely that the most complex problems in the equitable administration of ground-water legislation will be encountered where watercourse problems are found.

All of these situations, and additional ones, must be carefully considered before reaching conclusions and embodying them in legislation. Inherently, however, in contrast to present laws based on "safe yield" doctrine, it will be possible for workable legislation to be passed. Useful decisions can be made on the basis of the approach outlined previously, decisions that do not contradict the facts of hydrogeology.

CONCLUSIONS

It can be concluded that much of the presently existing body of ground-water law, law that is predicated on determining the "safe yield and annual recharge" (as in Oklahoma), will be found to be administratively unworkable, inequitable, and an obstacle that prevents society from achieving maximum water conservation and development. This conclusion rests on the fact that the concept of "safe yield" is a fallacious one.

It may also be concluded that much of the existing ground-water law will have to be rewritten to eliminate the concept of "safe yield" and to bring laws into accord with the facts of hydrogeology before real progress can be achieved in the conservation and development of our water resources through the use of aquifers. The cautious approach by most state legislatures in the field of ground-water law seems to have been fully justified.

Future legislation concerning ground water should be based on appropriation doctrine and applied with proper consideration for present techniques and probable future advances in the field of ground-water engineering. In particular, consideration should be given to the possibility of a technical

11. Thomas, H. E., Conservation of Ground Water: McGraw Hill, New York, 1951, p. 7.

breakthrough on the field of artificial recharge. Legislation should be written broadly enough to take advantage of this development when it occurs.

Finally, every effort should be made by professionals in the field of hydrogeology to remove the concept of "safe yield" from legislation, to eliminate it as an objective of ground-water studies, and to restudy all reports purporting to fix such a figure with the object of revising and reissuing such reports more in accord with determinable fact.

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EVAPORATION FROM FREE WATER SURFACES AT HIGH ALTITUDES[1]

Harry F. Blaney,[2] M. ASCE
(Proc. Paper 1104)

SYNOPSIS

In western United States, evaporation losses from reservoirs and lakes at high altitudes are of importance as an element affecting the net water supply available for irrigation crops, production of power, and municipal and industrial purposes. Except in unusual instances, evaporation cannot be measured directly from large water areas. Thus, it is common practice to measure evaporation from pans and use coefficients to reduce pan evaporation to lake evaporation. At high altitudes it is seldom possible to measure evaporation during the winter months because the water in the pans freezes. This paper presents data on evaporation in several western states and develops a method of estimating monthly evaporation for the entire year from temperature and other data.

INTRODUCTION

The storage of stream flow from mountain watersheds in reservoirs has made possible the development of much of the irrigated agriculture of the West. These reservoirs help to prevent floods, conserve a water supply that otherwise might be wasted, and make possible the production of power. The importance of a knowledge of water lost through evaporation to the efficient design and later operation of the works involved in a water-supply project has long been recognized by engineers.

There are very few instances where evaporation can be measured directly from lakes or reservoirs because of the unknown elements of supply; such as,

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1. Presented at a Regional Meeting of the ASCE, Irriga. and Drainage Div., September 8, 1955, Denver, Colo.
2. Prin. Irrig. Eng., Western Soil and Water Management Section, Soil and Water Conservation Research Branch, Agricultural Research Service, U. S.D.A., Los Angeles, Calif.

losses of water entering or leaving the reservoir. Thus, research studies have been necessary to determine the relationships existing between evaporation from standard pans and climatological data, which are measurable, and from lakes and reservoirs for which direct measurements are impossible. Most of the early studies on these relationships were made by the irrigation engineers of the United States Department of Agriculture in Colorado and California and coefficients were developed for reducing pan records to lake evaporation.[1,2,3,4] From April, 1950, to August, 1951, a comprehensive interagency evaporation experiment was conducted by the departments of Commerce, Interior, and Navy at Lake Hefner, Oklahoma.[5] In 1955, the U. S. Weather Bureau reported the results of a study on evaporation from pans and lakes.[6]

While evaporation pan records at low elevations are available for many areas throughout the United States, data on evaporation at high altitudes are less numerous and measurements are limited to frost-free months because the water in the pans freezes during the winter period. Evaporation losses during the winter months are needed for estimating water supply available from reservoirs during the summer months for irrigation, municipal and power purposes. For several years irrigation engineers of the United States Department of Agriculture have been conducting evaporation studies at high altitudes in the Huntington Lake area in California in cooperation with the Southern California Edison Company and evaporation from pans is being correlated with temperature observations for the purpose of estimating monthly evaporation for the entire year.

The primary purpose of this paper is to present data on evaporation from free water surfaces at high altitudes in California, Colorado, New Mexico, and Utah and to outline a procedure for estimating evaporation from climatological data where no pan records are available.

Effect of Altitude on Evaporation

A review of the literature indicates there is a difference of opinion among experimenters on the effect of altitude or barometric pressure on evaporation. Field observations show that temperature decreases as altitude increases and indicate that evaporation follows temperature more closely than altitude. Direct observations on the effect of altitude on evaporation, while the other conditions remain the same, are difficult to obtain.

In 1905 the late Samuel Fortier (M. ASCE) and Frank Adams of the Irrigation Investigations, U. S. Department of Agriculture, in cooperation with the State of California made observations to determine the effect of altitude on evaporation from a water surface by measuring the depth of water vaporized from a series of pans, 22 inches in diameter set on the ground at different elevations on the eastern slope of Mt. Whitney, California.[7] Table 1 shows the mean daily pan evaporation and temperature for a 20-day period.

The results show the decrease in evaporation to be more closely related to change in temperature than to change in barometric pressure. The close relationship of evaporation and temperatures is shown in Figure 1. Evaporation decreased uniformly from elevation 4,515 to 8,370 feet and more rapidly from there on up to 12,000 feet. At the summit of Mt. Whitney the evaporation

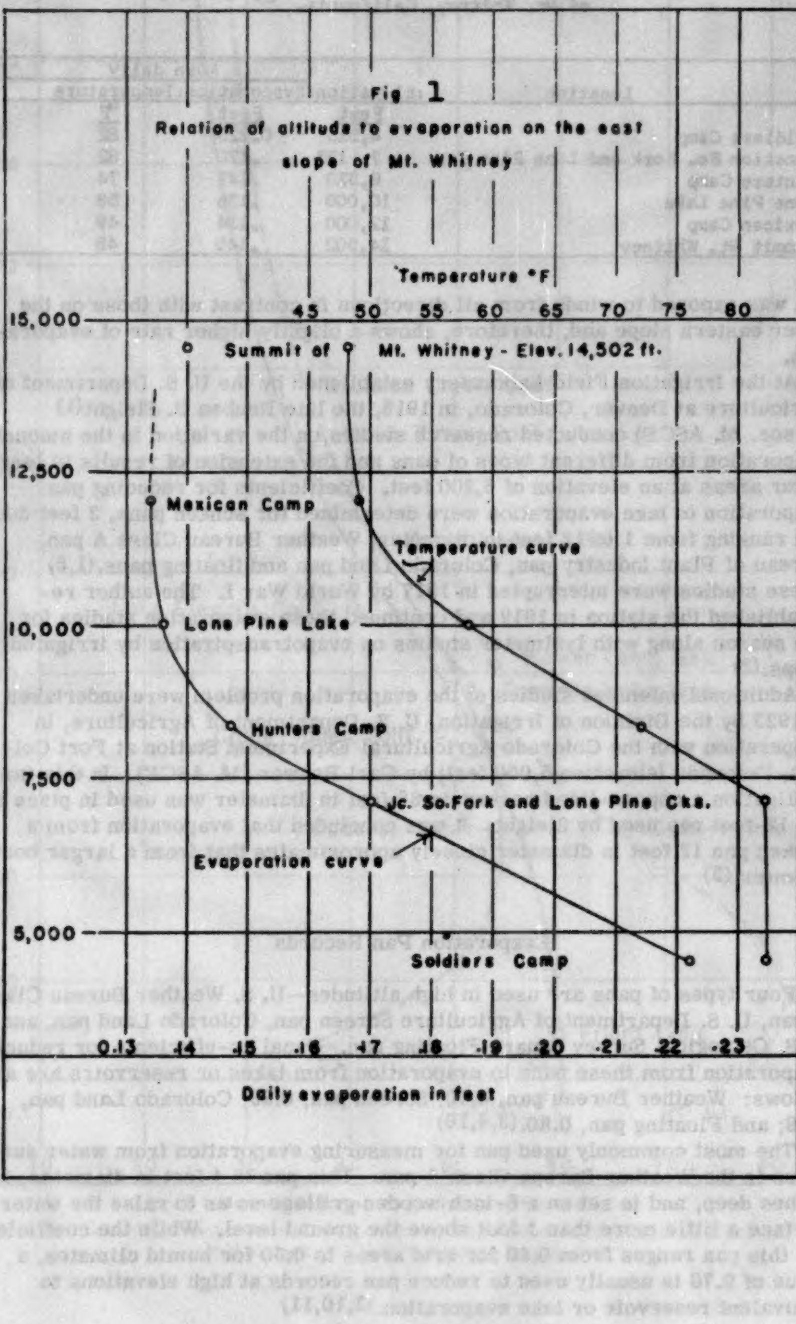
Fig 1
Relation of altitude to evaporation on the east slope of Mt. Whitney
Temperature °F
45 50 55 60 65 70 75 80
15,000
12,500
10,000
7,500
5,000
Elevation in feet
Summit of Mt. Whitney - Elev. 14,502 ft.
Mexican Camp
Temperature curve
Lone Pine Lake
Hunters Camp
Jc. So. Fork and Lone Pine Cks.
Evaporation curve
Soldiers Camp
0.13 .14 .15 .16 .17 .18 .19 .20 .21 .22 .23
Daily evaporation in feet

Table 1. - Evaporation and temperatures on the east slope of Mt. Whitney, California.

Location	Elevation	Mean daily Evaporation	Mean daily Temperature
	Feet	Feet	°F
Soldiers Camp	4,515	0.223	82
Junction So. Fork and Lone Pine Cr.	7, 125	.170	82
Hunters Camp	8,370	.147	74
Lone Pine Lake	10,000	.136	58
Mexican Camp	12,000	.134	49
Summit Mt. Whitney	14,502	.140	48

pan was exposed to winds from all directions in contrast with those on the lower eastern slope and, therefore, shows a slightly higher rate of evaporation.

At the Irrigation Field Laboratory established by the U. S. Department of Agriculture at Denver, Colorado, in 1915, the late Reuben B. Sleight(1) (Assoc. M. ASCE) conducted research studies on the variation in the amount evaporation from different types of pans and the extension of results to large water areas at an elevation of 5,200 feet. Coefficients for reducing pan evaporation to lake evaporation were determined for sunken pans, 3 feet deep and ranging from 1 to 12 feet in diameter; Weather Bureau Class A pan, Bureau of Plant Industry pan, Colorado Land pan and floating pans.(1,8) These studies were interrupted in 1917 by World War I. The author re-established the station in 1919 and continued these evaporation studies for one season along with lysimeter studies on evapotranspiration by irrigated crops.(2)

Additional intensive studies of the evaporation problem were undertaken in 1923 by the Division of Irrigation, U. S. Department of Agriculture, in cooperation with the Colorado Agricultural Experiment Station at Fort Collins, Colorado (elevation 5,000 feet) by Carl Rohwer (M. ASCE). In this investigation a copper-lined reservoir 85 feet in diameter was used in place of the 12-foot pan used by Sleight. It was concluded that evaporation from a sunken pan 12 feet in diameter closely approximates that from a larger body of water.(3)

Evaporation Pan Records

Four types of pans are used in high altitudes—U. S. Weather Bureau Class A pan, U. S. Department of Agriculture Screen pan, Colorado Land pan, and U. S. Geological Survey Square Floating pan. Usual co-efficients for reducing evaporation from these pans to evaporation from lakes or reservoirs are as follows: Weather Bureau pan, 0.70; Screen pan, 0.98; Colorado Land pan, 0.78; and Floating pan, 0.80.(3,4,10)

The most commonly used pan for measuring evaporation from water surfaces is the Weather Bureau Class A pan. This pan is 4 feet in diameter, 10 inches deep, and is set on a 6-inch wooden grillage so as to raise the water surface a little more than 1 foot above the ground level. While the coefficient for this pan ranges from 0.60 for arid areas to 0.80 for humid climates, a value of 0.70 is usually used to reduce pan records at high elevations to equivalent reservoir or lake evaporation.(3,10,11)

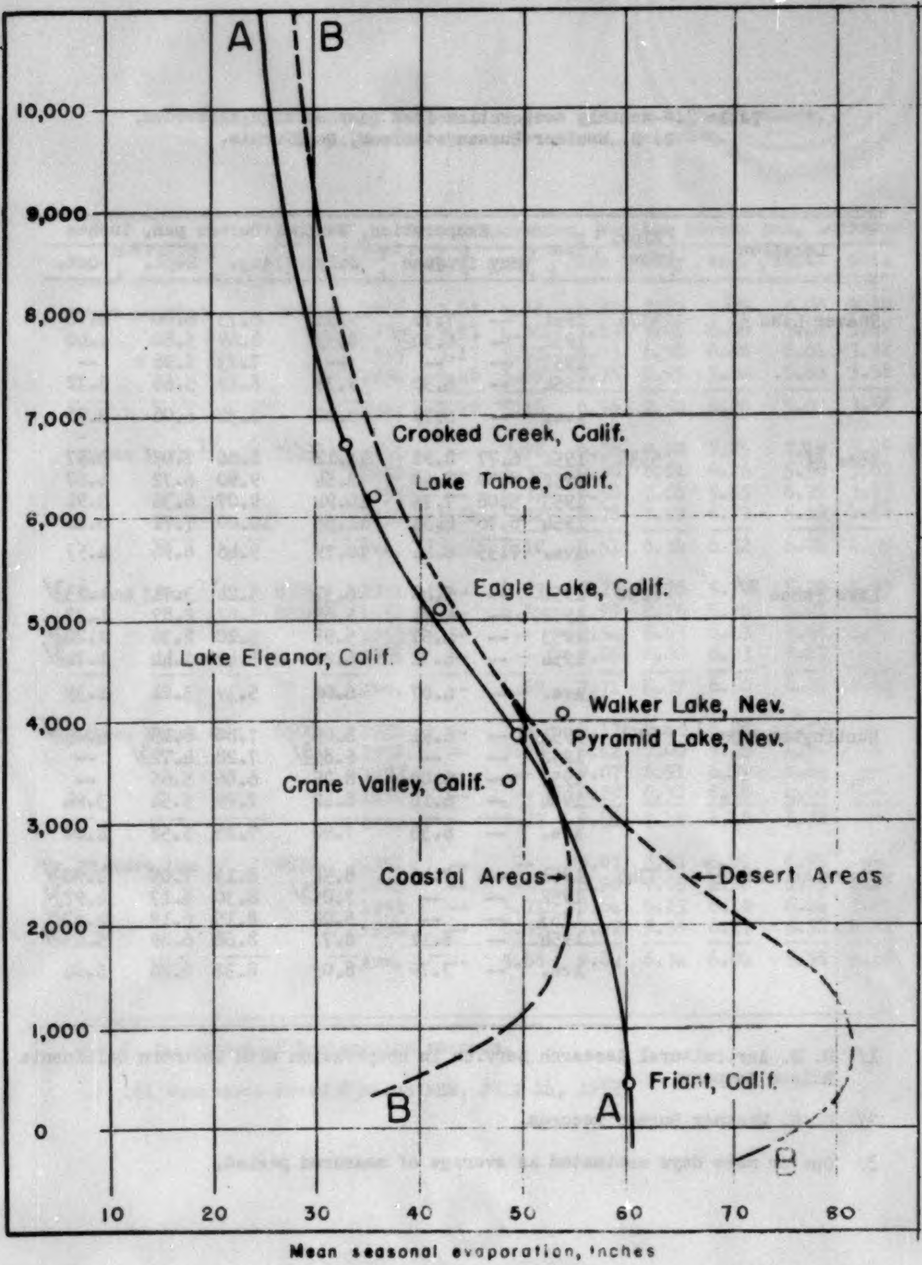

Figure 2 - Estimated mean annual evaporation from lake surfaces at different altitudes in Sierra Nevada Mountains

Table 2.--Monthly evaporation from pans at high altitudes, U. S. Weather Bureau stations, California.

Location	Elev. feet	Year	Evaporation, Weather Bureau pan, inches					
			May	June	July	Aug.	Sept.	Oct.
Shaver Lake 1/	5376	1951	--	7.78	9.31	8.73	6.60	3.76
		1952	--	6.30 3/	8.33	8.69	5.80	4.40
		1953	--	--	--	7.73	5.96	--
		1954	--	6.30	9.36	8.13	5.88	3.72
		Ave.	--	6.79	9.00	8.32	6.06	3.96
Boca 2/	5536	1951	6.77	8.98	11.12	8.86	7.03	3.57
		1952	8.79	8.68	9.54	9.90	6.72	4.67
		1953	5.08	7.75	10.94	9.07	6.36	3.91
		1954	8.76	8.34	11.55	10.00	7.72	6.11
		Ave.	7.35	8.44	10.79	9.46	6.96	4.57
Lake Tahoe 2/	6230	1951	--	5.48	6.37	5.24	3.71	1.23 3/
		1952	--	4.71	5.65	5.62	2.89	1.37
		1953	--	4.57	5.95	5.20	2.78	1.20 3/
		1954	--	4.72	6.26	5.49	3.44	1.74 3/
		Ave.	--	4.87	6.06	5.39	3.21	1.39
Huntington Lake 1/	6954	1951	--	6.91	8.05	7.80	6.15	4.05 3/
		1952	--	--	6.89 3/	7.28	4.72 3/	--
		1953	--	6.09 3/	8.25	6.86	5.65	--
		1954	--	6.14	8.42	7.09	5.54	3.94
		Ave.	--	6.38	7.90	7.26	5.52	4.00
Florence Lake 1/	7345	1951	--	7.46	8.54	8.13	7.05	4.90 3/
		1952	--	--	7.05 3/	8.30	5.17	4.97 3/
		1953	--	--	8.04	8.15	6.19	4.87 3/
		1954	--	8.12	8.71	8.68	6.69	5.43 3/
		Ave.	--	7.79	8.09	8.32	6.28	5.04

1/ U. S. Agricultural Research Service in cooperation with Southern California Edison Company.

2/ U. S. Weather Bureau records.

3/ One or more days estimated as average of measured period.

Table 3.--Monthly evaporation from pans at high altitudes, U. S. Weather Bureau stations, Colorado.

Location	Elev. feet	Year	Evaporation, Weather Bureau pan, inches						
			April	May	June	July	Aug.	Sept.	Oct.
Fort Collins	5004	1951	3.67	4.88	4.92	7.03	5.09	5.06	3.30
		1952	3.83	4.60	7.83	7.05	6.06	5.46	3.42
		1953	3.17	5.53	6.73	6.96	6.68	6.61	3.92
		1954	6.48	5.69	8.35	8.95	7.20	5.63	3.58
		Ave.	4.29	5.18	6.96	7.50	6.26	5.69	3.56
Estes Park 1/	7525	1951	--	7.98	6.38	9.10	7.25	7.19	3.98
		1952	--	6.32	10.30	8.61	6.26	5.50	3.62
		1953	--	4.79	8.58	7.64	5.85	6.29	3.53
		1954	5.54	5.32	8.78	7.99	6.73	5.19	4.88
		Ave.	5.54	6.10	8.51	8.34	6.52	6.04	4.00
Grand Lake 1/	8389	1951	--	4.73	5.27	7.76	5.78	5.20	1.63
	8288 2/	1952	--	4.64	7.99	8.76	5.94	6.01	--
		1953	--	--	8.54	8.43	6.13	7.01	3.65
		1954	--	--	9.05	8.60	8.53	5.87	--
		Ave.	--	4.69	7.71	8.39	6.60	6.02	2.64
Wagon Wheel Gap	8500	1951	--	--	9.13	8.48	6.08	6.67	--
		1952	--	7.22	9.42	7.00	5.35	4.90	--
		1953	--	--	9.07	6.91	6.39	6.64	--
		1954	--	--	10.20	6.32	6.58	4.66	--
		Ave.	--	7.22	9.46	7.18	6.10	5.72	--
Platoro Dam 1/	9826	1951	--	--	9.07	8.34	6.81	6.95	--
		1952	--	--	8.96	7.05	4.98	4.95	4.17
		1953	--	5.71	8.64	6.23	6.10	6.44	3.29
		1954	--	6.05	9.47	5.35	6.27	5.58	4.21
		Ave.	--	5.88	9.04	6.74	6.04	5.98	3.89

1/ U. S. Bureau of Reclamation records.

2/ All equipment moved 2 miles SSW, July 14, 1952.

Table 4.--Monthly evaporation from pans at high altitudes, U. S. Weather Bureau stations, New Mexico.

Location	Elev. feet	Year	April	May	June	July	Aug.	Sept.	Oct.	Nov.
			Evaporation, Weather Bureau pan, inches							
El Vado Dam	6750	1951	--	--	11.18	10.86	--	--	--	--
		1952	--	7.30	9.46	9.53	7.88	6.49	5.20	--
		1953	--	--	--	--	--	7.47	5.40	--
		1954	--	--	10.12	8.17	6.98	5.81	--	--
		Ave.	--	7.30	10.25	9.52	7.43	6.59	5.30	--
Santa Fe	7045	1951	--	9.91	12.17	10.66	8.63	9.68	5.74	2.58
		1952	--	9.34	12.52	10.58	--	8.99	7.37	--
		1953	--	10.51	13.27	11.74	10.55	10.33	--	--
		1954	8.70	8.40	11.78	8.69	8.23	7.10	5.53	--
		Ave.	8.70	9.54	12.44	10.42	9.14	9.03	6.21	2.58
Eagle Nest	8240	1951	--	--	--	9.32	--	7.46	--	--
		1952	--	--	--	8.84	6.59	5.66	--	--
		1953	--	--	--	6.98	--	6.03	4.30	--
		1954	--	--	11.11	5.82	5.45	6.00	5.24	--
		Ave.	--	--	11.11	7.74	6.02	6.29	4.77	--

Table 5.--Monthly evaporation from pans at high altitudes, U. S. Weather Bureau stations, Utah.

Location	Elev. feet	Year	Evaporation, Weather Bureau pan, inches						
			April	May	June	July	Aug.	Sept.	Oct.
Greenriver Airway	4063	1951	--	--	9.60	10.46	7.81	5.77	3.39
		1952	--	8.13	9.11	8.60	7.08	5.60	3.90
		1953	--	7.73	10.58	8.81	7.41	6.05	3.24
		1954	--	7.96	8.53	9.46	9.08	5.20	3.51
		Ave.	--	7.94	9.46	9.33	7.85	5.66	3.51
Utah Lake Lehi	4497	1951	5.64	7.49	8.69	10.38	8.86	7.12	3.20
		1952	5.30	8.65	10.46	9.80	9.15	7.59	4.56
		1953	5.11	6.50	10.24	10.32	9.20	6.47	3.97
		1954	7.21	8.60	9.19	10.97	10.22	7.02	3.79
		Ave.	5.82	7.81	9.65	10.37	9.36	7.05	3.88
Logan	4608	1951	5.65	6.56	7.28	8.61	7.86	6.36	2.48
		1952	--	7.29	9.03	9.19	8.17	6.32	3.88
		1953	3.57	4.53	7.70	8.77	9.53	5.93	3.50
		1954	5.39	7.13	6.56	9.61	9.34	6.11	3.22
		Ave.	4.87	6.38	7.64	9.05	8.73	6.18	3.27
East Portal	7608	1951	--	6.49	6.42	8.00	5.52	6.12	2.40
		1952	--	--	7.79	6.96	7.06	5.51	4.30
		1953	--	--	9.44	8.18	6.68	5.87	2.73
		1954	--	--	6.98	8.07	7.53	5.17	--
		Ave.	--	6.49	7.65	7.80	6.70	5.67	3.14

Observations of evaporation from Weather Bureau type pans were initiated some 50 years ago and this pan is now used throughout United States and Mexico and in other parts of the World. In western United States measurements are made by the Agricultural Research Service, Bureau of Reclamation, Weather Bureau, State Agricultural Experiment Stations in cooperation with other agencies and some of the results are published by the Weather Bureau. Results of some of these monthly measurements at high altitudes in California, Colorado, New Mexico, and Utah are shown in tables 2, 3, 4, and 5.

California Study in Huntington Lake Area

In 1946 studies of evaporation at high altitudes in lakes in the Sierra Nevada Mountains near Fresno, California, were started by the Division of Irrigation and Water Conservation,[4] Soil Conservation Service, U. S. Department of Agriculture in cooperation with the Southern California Edison Company. These investigations were under the supervision of the author for the government and William A. Land and Leonard L. Longacre for the company. Class A Weather Bureau Evaporation Stations equipped with Young Screen pans were installed at Shaver Lake (elev. 5,376 feet); Huntington Lake (elev. 6,954 ft.); Florence Lake (elev. 7,345 ft.); and Kaiser Pass (elev. 9,194 ft.) in the Upper San Joaquin River Watershed. The water from these lakes, after being used several times for developing electric power by the Southern California Edison Company, flows into a storage reservoir created by Friant Dam built by the U. S. Bureau of Reclamation and is available for irrigation use by the farmers in the San Joaquin Valley. Evaporation from a Class A Weather Bureau pan and temperatures have been measured by the Bureau of Reclamation at Friant (elev. 380 ft.) for many years.

The purpose of this investigation is to determine the monthly and annual evaporation losses from lakes at altitudes ranging from 4500 to 9200 ft. As indicated in table 2, measurements of pan evaporation during the winter months is not possible due to the freezing of water in the pans at these high elevations. Thus, the problem of determining evaporation during this period is rather difficult.

However, by correlating measured monthly evaporation (e) with mean monthly temperatures (t) and percent of daytime hours (p) for a five-month period, June to October, at Huntington Lake, evaporation for other months may be estimated as illustrated in table 6 and figure 3. This procedure is a modification of a method developed by Blaney and Morin in the Pecos River Joint Investigation in 1940-41 from evaporation records and related meteorological data at stations in New Mexico and Texas.(12,13) By multiplying the mean monthly temperature (t) by the monthly percentage of daytime hours of the year (p), a monthly use factor (f) is obtained. Then it is assumed that the monthly evaporation (e) varies directly as this factor. Expressed mathematically, $e = kf$ = monthly evaporation in inches, where $f = \frac{txp}{100}$ and k = monthly empirical coefficient computed from measured evaporation and temperature and percent of daytime hours. The relation between mean monthly evaporation (e) and water use factor (f) for the period 1947 to 1954 inclusive at Huntington Lake is shown in figure 3. This is based on data given in table 6.

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4. Transferred to Western Soil and Water Management Section, ARS, January 1, 1954.

Table 6.--Mean monthly observed and estimated evaporation from Weather Bureau Pan and estimated lake evaporation, 1947-54, Huntington Lake, California.

Month	t	p	f	Evaporation, inches Pan (e)	Evaporation, inches Lake 2/
June	51.5	9.89	5.09	6.67	4.67
July	60.8	10.05	6.11	8.18	5.73
August	58.3	9.44	5.50	7.33	5.13
September	56.5	8.37	4.73	5.61	3.93
October	48.0	7.82	3.75	3.62	2.53
November	40.4	6.87	2.77	2.1 1/	1.46
December	32.6	6.72	2.19	1.1 1/	.78
January	29.5	6.93	2.04	.9 1/	.63
February	30.9	6.82	2.11	1.0 1/	.71
March	30.0	8.35	2.51	1.7 1/	1.19
April	38.8	8.87	3.44	3.2 1/	2.24
May	44.6	9.87	4.40	5.1 1/	3.56
Total				46.51	32.56

t = Mean monthly temperature F°.

p = Monthly percent of daytime hours.

$f = \frac{t \times p}{100}$ = Monthly use factor

1/ Estimated from curve.

2/ Reduction coefficient 0.70.

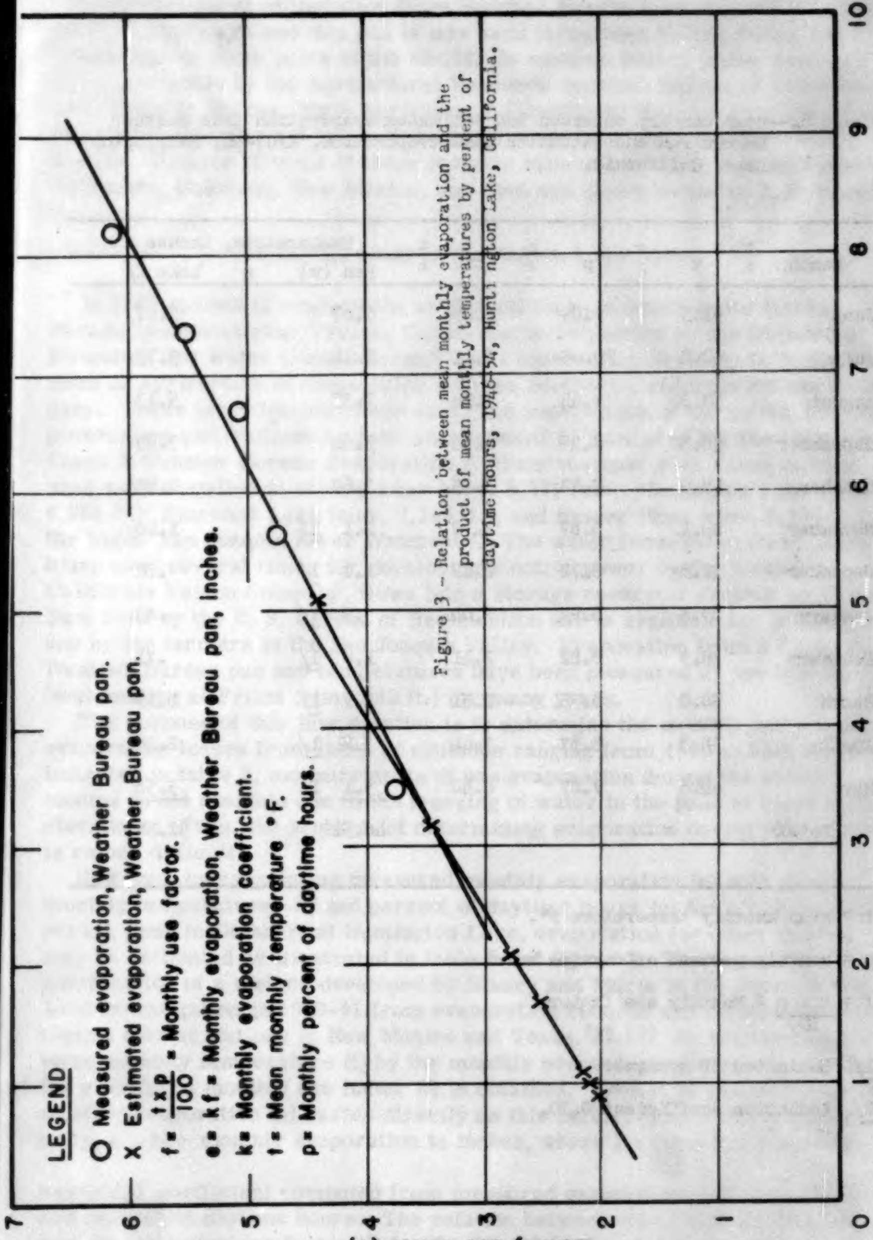

Figure 3 - Relation between mean monthly evaporation and the product of mean monthly temperatures by percent of daytime hours, 1946-54, Huntington Lake, California.

Table 7.--Estimated monthly evaporation from free water surfaces based on measured pan evaporation and temperature.

Location	Month	t	p	f	e	k	Sum of f	Sum of e	Lake evaporation
		F°	Percent					Inches	Inches1/
Colorado									
Fort Collins									
Elev. 5004 ft.	Jan.	30.4	6.72	2.04	1.432/	0.70	2.04	-	1.00
	Feb.	34.0	6.71	2.28	1.822/	.80	4.32	-	1.27
	Mar.	34.7	8.32	3.11	2.482/	.80	7.43	-	1.74
	Apr.	46.0	8.97	4.13	4.29	1.04	11.56	4.29	3.00
	May	54.9	10.05	5.52	5.18	.94	17.08	9.47	3.63
	June	65.6	10.11	6.63	6.96	1.05	23.71	16.43	4.87
	July	71.6	10.26	7.35	7.50	1.02	31.06	23.93	5.25
	Aug.	68.4	9.56	6.54	6.28	.96	37.60	30.19	4.40
	Sept.	61.3	8.39	5.14	5.71	1.11	42.74	35.88	4.00
	Oct.	49.2	7.73	3.80	3.57	.94	46.54	39.44	2.50
	Nov.	36.4	6.70	2.44	1.952/	.80	48.98	-	1.36
	Dec.	29.7	6.48	1.92	1.342/	.70	50.90	-	.94
	Total				48.51				33.96
Utah									
Logan									
Elev. 4608 ft.	Jan.	27.9	6.62	1.85	1.292/	0.70	1.85	-	0.90
	Feb.	29.3	6.65	1.95	1.562/	.80	3.80	-	1.09
	Mar.	34.7	8.31	2.88	2.302/	.80	6.68	-	1.61
	Apr.	47.5	9.00	4.28	4.87	1.14	10.96	4.87	3.41
	May	54.3	10.14	5.51	6.39	1.16	16.47	11.25	4.47
	June	61.5	10.21	6.28	7.66	1.22	22.75	18.89	5.36
	July	71.1	10.35	7.36	9.05	1.23	30.11	27.94	6.33
	Aug.	69.6	9.62	6.70	8.71	1.30	36.81	36.67	6.10
	Sept.	61.7	8.40	5.18	6.16	1.19	41.99	42.85	4.31
	Oct.	50.1	7.70	3.86	3.28	.85	45.85	46.12	2.30
	Nov.	37.1	6.62	2.46	1.972/	.80	48.31	-	1.38
	Dec.	26.4	6.38	1.68	1.182/	.70	49.99	-	.83
	Total				54.42				38.09

t = Mean monthly temperatures 1951-1954 inclusive.
p = Monthly percent of daytime hours of the year.
f = $\frac{t \times p}{100}$ = Monthly use factor.
e = kf = Mean monthly evaporation Weather Bureau pan, inches.
k = $\frac{e}{f}$ = Monthly empirical use coefficient.

1/ Estimated by reduction factor 0.70.
2/ Estimated k x f

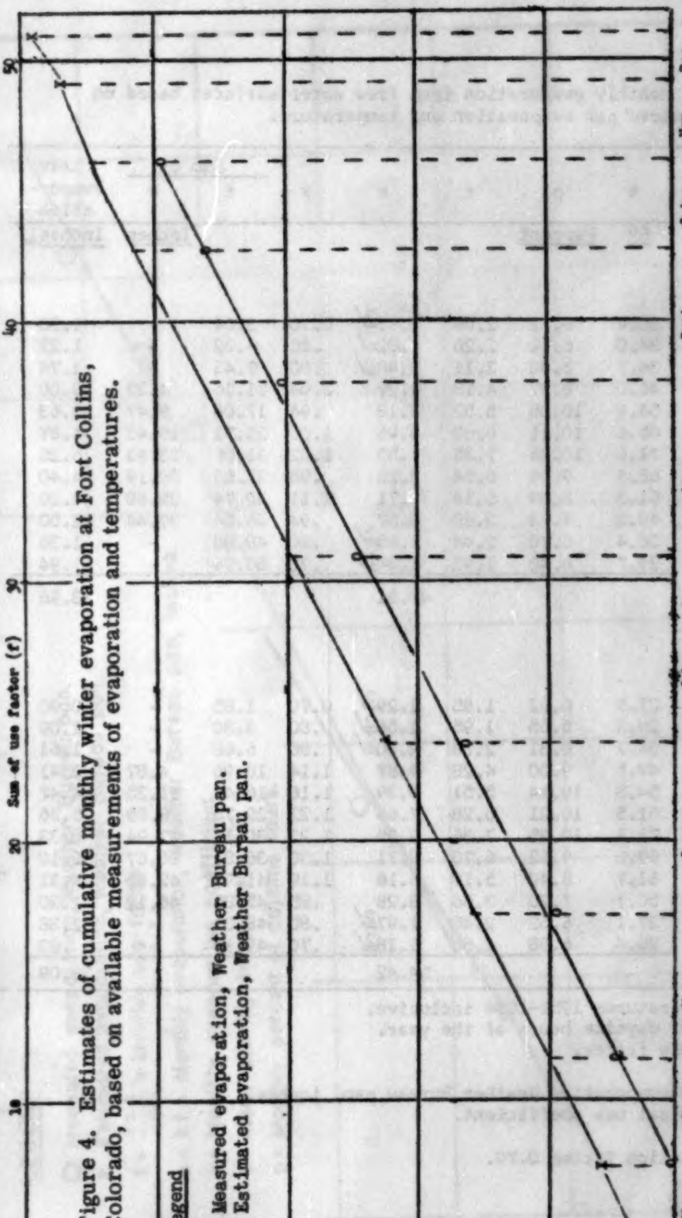

Figure 4. Estimates of cumulative monthly winter evaporation at Fort Collins, Colorado, based on available measurements of evaporation and temperatures.

Evaporation Formula

$e = kf$ = monthly evaporation inches
k = monthly emperical coefficient
$f = \frac{txp}{100}$ = monthly use factor
t = mean monthly temperature F°
p = monthly percent of daytime hours of the year

An analysis of data at stations in this area and for stations at lower elevations show that the curve in figure 3 may be plotted as a straight line for the period April to October and for the winter months the curve will be an approximate straight line at a different slope for monthly pan evaporation of one inch or more. The monthly evaporation for November to May is estimated from figure 3 as shown in table 6.

At some of the stations, measurements are being made of wind movement, humidity, and water temperatures as well as pan evaporation and air temperatures. Analyses of these data have not been completed.

LITERATURE CITED

1. Evaporation from the Surfaces of Water and Riverbed Materials. By R. B. Sleight. Jour. Agric. Res. Vol. 10, No. 5. 1917.
2. Progress Report on Evaporation and Use of Water Studies at Irrigation Field Laboratory, Denver, Colorado. By Harry F. Blaney, U. S. Dept. of Agric. 1920. (Unpublished)
3. Evaporation from Free Water Surfaces. By Carl Rohwer. Tech. Bul. No. 271. U. S. Dept. of Agri. 1931.
4. Evaporation from Water Surfaces in California. By Arthur A. Young. U. S. Dept. of Agri. Bul. Nos. 54 and 54-A, Calif. Division of Water Resources. 1947.
5. Water-Loss Investigations: Volume I - Lake Hefner Studies. U. S. Geological Survey Professional Paper No. 269. 1954.
6. Evaporation from Pans and Lakes. By M. A. Kohler, T. J. Nordensen, and W. E. Fox. U. S. Weather Bureau Research Paper No. 38. 1955.
7. Evaporation Losses in Irrigation. By Samuel Fortier. Eng. News, Vol. 58. 1907.
8. Discussion of Paper: Evaporation on United States Reclamation Projects. By R. B. Sleight. Trans. ASCE, Vol. 90. 1927.
9. Evaporation from Large Water Surfaces Based on Records in California and Nevada. By S. T. Harding. Nat. Res. Council, Trans. Am. Geophys. Union. 1935.
10. Standard Equipment for Evaporation Stations. Final Report of Sub-Committee on Evaporation of the Special Committee on Irrigation Hydraulics. Trans. ASCE, Vol. 99. 1934.
11. Evaporation from Water Surfaces in California. By Harry F. Blaney and Gilbert L. Corey. Bul. No. 54-B, Division of Water Resources, State of California. 1955.
12. Consumptive Water Use and Requirements. By Harry F. Blaney, Paul A. Ewing, Karl V. Morin, and Wayne D. Criddle. Part III, Section 3. Pecos River Joint Investigations. National Resources Planning Board. 1942.
13. Consumptive Use of Water—Definitions, Methods, and Research Data. By Harry F. Blaney. Trans. ASCE, Vol. 117. 1952.

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IRRIGATION REQUIREMENTS BASED ON CLIMATIC DATA[a]

George H. Hargreaves,[1] A.M. ASCE
(Proc. Paper 1105)

SYNOPSIS

Climatic data have long been used in the computation of consumptive use and irrigation requirements. This paper shows the limitations of present methods. The evaporation of water is considered as a physical process. Physical laws, climatic data and theoretical considerations are used in the derivation of new equations for determining the consumptive use or evapo-transpiration potential for any set of climatic conditions. A formula, based upon the use of evapo-transpiration potentials, is developed for transferring consumptive use data from one set of climatic conditions to another. Climatic regions for the United States are described and the use of consumptive use data in computing irrigation requirements is discussed. This paper provides the irrigation engineer the data required for the computation of consumptive use for any given set of climatic and cropping conditions.

INTRODUCTION

The need for improved irrigation efficiencies is much greater now than ever before. With some 26,000,000 acres under irrigation in 19 Western States it has been necessary to collect much data concerning irrigation requirements. Water shortages occur each year on about half of this area while large areas receive excessive irrigation supplies thereby raising the water table and in many instances creating salinity and drainage problems.[2] Even

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a. Presented at a meeting of the Irrigation and Drainage Division, ASCE. Sept. 8-10, 1955, Denver, Colo.

1. Civ. Engr., Inst. of Inter-American Affairs, International Cooperation Administration, Port-au-Prince, Haiti; presently with ICA, Manila, Philippine Islands.

2. Irrigation in the United States, by George D. Clyde, ASCE, Transactions Vol. CT, 1953, pages 338 and 339.

with the widespread shortages of irrigation water efficiencies are very low. Sufficient data are not available for accurate computation, but it is believed that irrigation efficiencies average less than 30 percent.

Irrigation in the United States is much more efficient than is the case in other countries. Making allowance for reasonable efficiency, crops rarely receive irrigation water in accordance with their consumptive use requirements. Large areas have been taken out of agricultural production due to a raising water table and increasing salinity. In Haiti important irrigated areas receive only a small fraction of the water required during the dry season. Often during periods of favorable rainfall these same lands are over-irrigated. Improvement of crop production can be realized by improving the knowledge of irrigation requirements for individual crops as well as through improving irrigation supplies.

Need for Improved Methods

During the past 40 years much data have been collected concerning the irrigation requirements of individual crops. These have proven valuable to the irrigation engineer working in the area where the data were collected.

Several formulas have been developed for the transfer of irrigation requirement or consumptive use data from one area to another. Probably the best known of these methods is the Blaney-Criddle formula.[3] This formula is based upon the assumption that consumptive use (or evaporation) varies with the mean monthly temperature (t), the extent of daytime hours expressed as monthly percentage of daytime hours of the year (p) and in the case of consumptive use, with available moisture. Seasonal or growing period consumptive use (U) is computed by multiplying a coefficient (K) by the summation of the products of t multiplied by p and divided by 100. By choosing a value of "K" for each particular climatic zone and for each crop a good computation of seasonal use can be made for most of the climatic conditions found in the United States. Limited use of the Blaney-Criddle formula has been made using monthly consumptive use coefficients.[4,5]

The Blaney-Criddle formula does not, however, except for areas where monthly coefficients are available, give the monthly distribution of consumptive use. Both in the preparation of operating schedules on irrigation projects and in the design of canal systems it is necessary to have a close approximation of monthly demands for irrigation.

Another important consideration is that, with the increased technical assistance and economic aid given by various agencies in the field of irrigation it is now necessary to transfer consumptive use requirements to many parts of the world. Larson,[6] working in the Philippines found that the Blaney-Criddle formula did not apply. Fuhriman and Smith[7] measured the

3. ASCE Transactions Vol. 117, 1952, pages 964, 965.
4. ASCE Transactions Vol. 117, 1952, page 971.
5. ASCE Proceedings Vol. 80, 1954, Separate No. 522, page 26.
6. Larson, Fred H., in Discussion of "Humid Area Soils and Moisture Factors for Irrigation Design" ASCE Proceedings Vol. 81, 1955, Separate No. 621, page 7.
7. Fuhriman, D. K., and Smith, R. M., Conservation and Consumptive Use of Water with Sugar Cane Under Irrigation in the South Coastal Area of Puerto Rice, The Journal of Agriculture of the University of Puerto Rico, Vol. XXXV, January 1951.

consumptive use by sugar cane in the south coastal area of Puerto Rico. These studies clearly indicate that in the computation of consumptive use by sugar cane additional factors other than mean monthly temperature and daylight hours must be considered.

Factors Influencing Evaporation

In the laboratory, evaporation of water is a physical process dependent upon energy to supply heat of vaporization and the removal of water vapor. Evaporation from a U. S. Weather Bureau pan, from a free water surface or from the stomata of plants consequently depend upon the energy input or energy balance and upon the factors which remove the evaporated or transpired vapor. Energy input has been correlated with length of day, sunshine, air temperatures and insolation in Langleys (gram-calories of energy per square centimeter). Of these measurable weather elements data are most readily available for air temperature and length of day. The most important climatic factors correlating with the removal of vapor are relative humidity and velocity of wind movement.

The relationship between evaporation, temperature and length of day can be reduced to an equation from theoretical considerations and climatic data. Evaporation is a function of the heat of vaporization and the length of time during which the heat is applied. Without considering the effect of other factors, heat of vaporization is measured by temperature in ^{o}C or temperature in ^{o}F above 32. A straight line relationship is indicated which can be expressed mathematically by the equation:

$$e = m\ (t - 32) \qquad (1)$$

in which e is monthly evaporation in inches; m is a factor denoting the slope of the straight line as influenced by those factors associated with the removal of water vapor and the length of time the temperature and other factors are applied; and t is the mean monthly temperature in ^{o}F.

Formula 1 can be refined by correcting for the time element and expressed as follows:

$$e = c\ d\ (t - 32) \qquad (2)$$

in which d is a monthly daytime coefficient and c is a climatic factor depending upon humidity and to a minor degree upon wind movement. Values of "d" are computed by relating monthly daytime hours for each month to an average value (a monthly percent of daytime hours of the year of eight and one third is equal to an index of 1.00). Monthly daytime coefficients calculated for latitudes 5 to 50 north are presented in Table 1.

By disregarding the effect of wind movement it can be assumed that the climatic factor (c) influencing the removal of water vapor is a function of mean monthly relative humidity. Relative humidity at noon seems to provide the most usable data. From theoretical considerations evaporation would approach zero as relative humidity at noon approaches 100. By assuming a straight line relationship and using climatic data the climatic factor (c) was evaluated as follows:

$$c = 0.38 - 0.0038\ h \qquad (3)$$

in which h is the mean monthly relative humidity at noon. Equation 2 becomes:

$$e = d\,(0.38 - 0.0038\,h)\,(t - 32) \qquad (4)$$

In the transfer of consumptive use or irrigation requirements from one location to another it is necessary to know the influence that can be expected from the weather. Evaporation from a free water surface or from a U. S. Weather Bureau pan is the best index of the consumptive use (evapo-transpiration) potential. Since evaporation measurements are frequently unavailable, it is necessary for the irrigation engineer to compute consumptive use potentials from other weather data.

The relationship between mean monthly relative humidity at noon and values of "c" is shown in Figure 1. From equation 2 and temperature, humidity and evaporation data at Davis, California, value of "c" were computed for each month of the year and plotted in Figure 1. Based upon data presented in the Climatic Atlas,[8] the United States was divided into five climatic regions each representing a different classification with respect to relative humidity during the period April thru October. These climatic classifications together with the corresponding ranges in the climatic factor (c) are shown in Figure 2. Due to the variations in climatic conditions along the Pacific Coast a climatic classification was not assigned to this area.

If the mean monthly relative humidity is known, the climatic factor (c) can be evaluated from Figure 1. Where data are not available the factor can be approximated from Figure 2.

In the use of climatic data it may be desirable to consider that for the principal locations used in evaluating the relationship between humidity and the climatic factor normal summer sunshine averaged about 70 percent of possible sunshine. Possibly some correction may be required for the areas along the Appalachian Mountains, in New England and along the eastern coast where summer cloudiness is increased. Also, the south eastern portion of California and southwestern Arizona have about 90 percent of possible summer sunshine. It may be necessary to increase values of "c" in this area.

Transfer of Consumptive Use Data

Transfer of consumptive use or irrigation requirement data from one climatic region to another can be accomplished in a manner similar to that developed by Blaney and Criddle.[3] Consumptive use for a specific crop is assumed to vary, under differing climatic conditions, directly with the consumptive use potential. Expressed mathematically:

$$U = KE = \Sigma\, k\, e \qquad (5)$$

in which U is the consumptive use of a crop in inches for the irrigation season or for a stated period; E is the sum of the monthly values of evaporation or consumptive use potential for the period; K is an emperical coefficient depending upon the individual crop grown; k is the monthly consumptive use

8. Climatic Atlas of the United States, by Stephen S. Visher, Harvard University Press, Cambridge, 1954.

(evapotranspiration) potential as previously defined. Monthly and growing season values of consumptive use coefficients from data at Davis, California, are shown in Table 2.

In tropical countries sugar cane and bananas are two of the more important commercial crops. Consumptive use data are available for sugar cane from research on the south coast of Puerto Rico.[7] Some data for bananas have been collected in the Dominican Republic and in Jamaica. Based on these data, monthly and seasonal consumptive use coefficients for sugar cane and bananas are given in Table 3.

Experiments by Fuhriman and Smith[7] clearly demonstrate real differences in consumptive use associated with rate and stage of the crop growth. Consumptive use not only depends upon the consumptive use potential but upon the availability of moisture and upon all of the climatic, soil and biological conditions that influence the rate and stage of plant growth. Plant characteristics and other conditions govern the amount of water reaching the plant stomata. In the use of the consumptive use coefficients shown in Tables 2 and 3 considerable care must be exercised in order to use rates and stages of growth that are comparable. If sufficient care is exercised in this then the data with respect to the consumptive use of water by the crops grown at Davis, California, or at a given location, can be used in computing consumptive use under similar cultural practices for any other location.

Irrigation Requirements

The procedures outlined above permit the computation of month by month consumptive use requirements for a given set of climatic conditions and cultural practices for a particular crop pattern. After computing the consumptive use requirements, the irrigation engineer must then compute or assume an irrigation efficiency as a basis for design.

Perrenial crops and also many annual crops have rates of consumptive use that are not influenced by the moisture level in the soil as long as the moisture is above the permanent wilting point. Principal exceptions are annual crops with root systems that occupy only a portion of the soil mass. For these crops with skeleton root systems, it seems desirable to irrigate when the wilting point is reached in the upper 30 percent of the crop root zone.

A primary responsibility for improving irrigation efficiencies rests with the irrigation engineer. In an under-developed country where the water users have little or no knowledge of irrigation requirements, high irrigation efficiencies can only be achieved if irrigation deliveries are limited to the consumptive use requirements plus an allowance to take into consideration reasonable irrigation efficiencies.

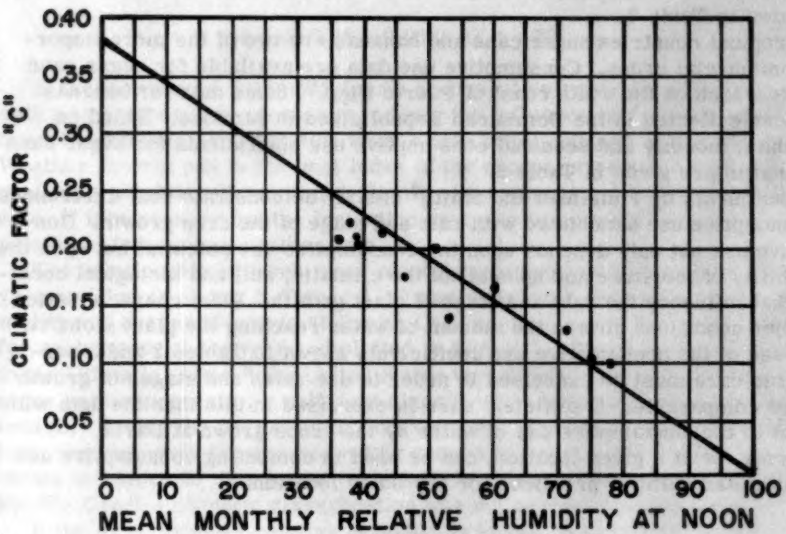

FIGURE 1
CLIMATIC FACTORS

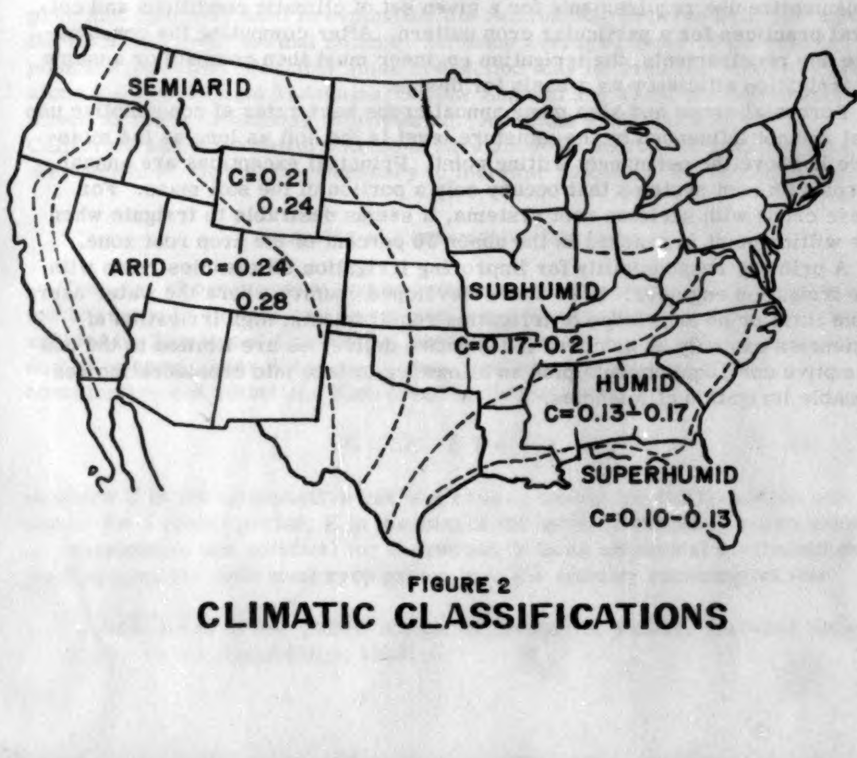

FIGURE 2
CLIMATIC CLASSIFICATIONS

TABLE 1

MONTHLY DAYTIME COEFFICIENTS (d)

North Latitude in Degrees	January	February	March	April	May	June	July	August	September	October	November	December
5	1.01	0.91	1.02	0.99	1.03	1.00	1.03	1.03	0.98	1.02	0.98	1.00
10	0.98	0.89	1.02	1.01	1.05	1.03	1.06	1.05	0.99	1.00	0.95	0.97
15	0.96	0.88	1.01	1.01	1.08	1.06	1.08	1.06	0.99	0.99	0.93	0.95
20	0.93	0.87	1.01	1.02	1.10	1.08	1.11	1.08	0.99	0.98	0.91	0.92
25	0.91	0.86	1.01	1.03	1.12	1.11	1.13	1.09	1.00	0.97	0.89	0.89
30	0.88	0.84	1.00	1.05	1.14	1.14	1.16	1.11	1.00	0.96	0.86	0.86
35	0.85	0.83	1.00	1.06	1.17	1.17	1.19	1.12	1.00	0.94	0.84	0.82
40	0.81	0.81	1.00	1.08	1.20	1.21	1.23	1.14	1.01	0.93	0.81	0.78
45	0.77	0.79	0.99	1.09	1.24	1.26	1.27	1.17	1.01	0.91	0.77	0.74
50	0.72	0.76	0.99	1.11	1.28	1.32	1.32	1.20	1.01	0.89	0.73	0.68

TABLE 2

MONTHLY AND SEASONAL CONSUMPTIVE USE COEFFICIENTS[a]
(Davis, California)

CROP	MONTHLY CONSUMPTIVE USE COEFFICIENTS "k"									SEASONAL COEFFICIENT "K"
	March	April	May	June	July	August	September	October	November	
Alfalfa	0.41	0.70	0.64	0.67	0.74	0.67	0.64	0.40	0.41	0.59
Almonds	0.16	0.36	0.34	0.52	0.48	0.34	0.29	0.48	0.21	0.36
Asparagus	0.16	0.11	0.12	0.18	0.46	0.81	0.84	0.99	0.51	0.46
Beans (Lima)				0.41	0.51	0.61	0.32			0.46
Beans				0.15	0.28	0.66	0.51			0.40
Cantaloupes			0.24	0.31	0.37	0.61	0.38			0.48
Carrots	0.16	0.18	0.19	0.52	0.64	0.28				0.33
Celery				0.15	0.14	0.25	0.45	0.70	0.85	0.42
Citrus	0.41	0.36	0.44	0.43	0.44	0.41	0.41	0.64	0.41	0.44
Corn				0.12	0.38	0.42	0.26	0.10		0.26
Fruit (deciduous)	0.14	0.45	0.49	0.74	0.71	0.55	0.43	0.36		0.48
Grain sorghums			0.07	0.30	0.39	0.30	0.15			0.24
Grain and Hay	0.50	0.75	0.58	0.12						0.49
Grapes (Muscat)		0.13	0.24	0.26	0.31	0.26	0.26	0.18		0.23
Hops		0.07	0.12	0.31	0.61	0.61	0.38			0.35

a. Based upon consumptive use data for Davis, California, published in "Suggested Subject Matter for Presentation at Irrigation Meetings" by L.J. Booher, 1948, College of Agricultural Extension Service, Davis, California. File 12.1.

TABLE 2 (Continued) - -

MONTHLY AND SEASONAL CONSUMPTIVE USE COEFFICIENTS
(Davis, California)

CROP	MONTHLY CONSUMPTIVE USE COEFFICIENTS "k"									SEASONAL COEFFICIENT "K"
	March	April	May	June	July	August	September	October	November	
Ladino Clover	0.50	0.81	0.55	0.77	0.83	0.76	0.70	0.44		0.67
Onions (early)	0.28	0.45	0.30							0.34
Onions (late)	0.28	0.45	0.30	0.31	0.28					0.32
Pasture	0.11	0.25	0.29	0.33	0.31	0.32	0.32	0.22	0.14	0.25
Peaches	0.22	0.45	0.43	0.46	0.51	0.51	0.38	0.60	0.41	0.44
Peas	0.28	0.36	0.49	0.31						0.36
Potatoes (early)	0.55	0.72	0.73	0.62						0.66
Prunes	0.17	0.34	0.34	0.50	0.48	0.32	0.42	0.48	0.24	0.37
Rice		0.32	1.34	1.42	1.40	1.44	0.51			1.07
Sudan Grass			0.24	0.33	0.37	0.35	0.28	0.24		0.30
Sugar Beets	0.19	0.27	0.55	0.87	0.69	0.36	0.15	0.10	0.03	0.36
Tomatoes				0.32	0.41	0.71	0.67	0.81		0.58
Walnuts		0.36	0.43	0.57	0.67	0.63	0.26	0.36	0.24	0.44
Watermelons				0.15	0.18	0.25	0.51			0.27

TABLE 3

CONSUMPTIVE USE COEFFICIENTS – SUGAR CANE AND BANANAS
(Caribbean Area)

CROP	MONTHLY CONSUMPTIVE USE COEFFICIENTS "k"												SEASONAL COEFFICIENT "K"
	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	
Sugar Cane*	0.77	0.69	0.49	0.52	0.53	0.56	0.59	0.73	0.85	0.84	0.91	0.86	0.70
Bananas	0.86	0.85	0.73	0.88	0.85	0.86	0.85	0.78	0.88	0.86	0.76	0.78	0.83

* Planted in March.

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DISCUSSION
(Proc. Paper 1111)

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Discussion of "WATER RIGHTS IN HUMID AREAS"

by Howard T. Critchlow
(Proc. Paper 705)

HOWARD T. CRITCHLOW,[1] M. ASCE.—The subject of water rights in the 31 Eastern States is becoming increasingly important in respect to agricultural use. The original paper and the discussions thereof emphasize the rapid growth both in acreage and number of farms on which supplemental irrigation is being practiced. More and more papers and articles are being published both in technical and popular literature on this subject.

The recent (1955) Yearbook of Agriculture, "Water" published by the United States Department of Agriculture contains a very interesting and informative chapter on "Water for Irrigation." A sub-section thereof on "Supplemental Irrigation in Humid Regions" (pages 252-258) by Max M. Tharp, assistant head, Southern Field Research Section, and C. W. Crickman, assistant head, Northern Field Research Section, Production Economic Research Branch, Agricultural Research Service, U.S.D.A., quote statistics from the U. S. Census on acreage irrigated and number of farms upon which irrigation is practiced in the 31 Eastern States in 1939 and 1949. These amounts are combined with those of the author's for 1954, and are given in the following table:

Supplemental Irrigation in 31 Eastern States

	1939	1949	1954
Acreage irrigated	739,000	1,517,000	2,418,000
Percentage of 1939	100	205	327
Number of farms	16,515	23,585	
Percentage of 1939	100	143	

This phenomenal growth in supplemental irrigation practice emphasizes the importance of individual irrigators and others recognizing the limitations of their water rights under the common law riparian doctrine which still governs in all Eastern States. Since water used for irrigation is a consumptive use and is not returned to the source as required under the riparian doctrine, but will reduce the amount available to lower riparian owners, said doctrine can be hydrologically sound only in an area of water surplus, as emphasized by H. E. Thomas(1) M. ASCE in his valued discussion. Dr. Thomas also points out that the increasing use of water for irrigation together with increasing demands by municipalities and industries will result in more critical shortages in many localities that are already short of water. This situation may lead to a modification of the riparian doctrine to clarify water rights and which uses of water have higher priority, and equally important encourage

1. Cons. Hydr. Engr., Trenton, N. J.

long-range planning in the conservation of water by storage reservoirs on surface streams and recharge of depleted underground reservoirs.

C. E. Busby(2) in his constructive discussion lists a number of eastern states that have improved their water policies in recent years which vary considerably from state to state; but more significant, few states have implemented their policies by legislation to control water use. Mr. Busby suggests, and the writer agrees, that water control legislation is a state problem and must be designed to meet local conditions and practices.

Paul H. Berg,(3) A.M. ASCE advises that the Kansas water law of 1945 governing the appropriation of water for beneficial use other than domestic is being contested in the Federal courts under the 14th amendment. The writer has intimate experience in administering water control laws in New Jersey which have been tested in the courts and upheld. It is hoped the Kansas law may be upheld in the interest of progressive water legislation.

J. C. Alexander(4) advises of technical delays in enacting legislation in Missouri to authorize a thorough study of the water resources of that state and the uses they should fulfill. Time, patience, and public education are necessary to overcome inertia and opposition to progressive ideas in long-range planning in water conservation and development. It took ten years to get ground water control legislation in New Jersey enacted in 1947 (Chapter 375). Mr. Alexander also revises his estimate of total area under irrigation for 1954 in Missouri in excess of 40,000 acres excluding rice production, instead of 2000 acres originally reported.

John R. Carreker, Superintendent, Soil and Water Conservation Research Branch, Agricultural Research Service, U.S.D.A., reports by letter that the Georgia Water Use and Conservation Committee was created by about 50 organizations through voluntary efforts in 1954. As a result of this activity, the General Assembly of Georgia in 1955 created a Georgia Water Law Revision Commission with the purpose of reviewing water laws and reporting back to the next session of the General Assembly. He also adds that the area under irrigation in 1954 was 27.701 acres, there being no amount reported in the table in the original paper.

Engineers should take a leading part in drafting legislation on water control and, when enacted, should be entrusted with a prominent part in the administration of such laws. One encouraging sign in that direction is the recent (1956) appointment of a Task Committee on Water Rights Laws in States in Humid Areas by the Irrigation and Drainage Division. One objective of this committee should be the preparation of specifications for a model law on water control at the state level.

REFERENCES

1. Staff Geologist, Ground Water Branch, U. S. Geological Survey, Salt Lake City, Utah.
2. Water Rights Specialist, Soil Conservation Service, Berkeley, Calif.
3. Project Manager, Kansas River Projects, Bureau of Reclamation, U. S. Department of the Interior, McCook, Nebr.
4. Water Engineer, Missouri Division of Resources and Development, Jefferson City, Mo.

Discussion of
"GENERAL ASPECTS OF PLANNED GROUND WATER UTILIZATION"

by Robert O. Thomas
(Proc. Paper 706)

ROBERT O. THOMAS,[1] A.M. ASCE.—The important contributions to the subject of the paper made by Drs. Thomas and Clendenen are appreciated. It is regretted that more extensive discussion of the problems involved, with particular reference to areas other than California, was not forthcoming.

The problem of determination of safe yield from an underground reservoir was discussed by Dr. Thomas. Evaluation of this aspect of ground water storage has heretofore been expressed (in California) as the maximum rate of extraction of water from a ground water basin which, if continued over an indefinitely long period of years, would result in the maintenance of certain desirable fixed conditions. This rate of extraction is determined by one or more of the following criteria:

1) Mean seasonal extraction of water from the ground water basin does not exceed mean seasonal replenishment to the basin.

2) Water levels are not so lowered as to cause harmful impairment of the quality of the ground water by intrusion of other water of undesirable quality, or by accumulation and concentration of degradents or pollutants.

3) Water levels are not so lowered as to imperil the economy of ground water users by excessive costs of pumping from the ground water basin, or by exclusion of users from a supply therefrom.

The criteria expressed above have usually, in the past, been applied to the natural hydrologic balance of inflow and outflow in individual ground water basins. Occasionally, they have been extended to include imported water supplies as an item of replenishment for basins where the annual draft is greatly in excess of the natural, or native replenishment. The next step will, of course, be the deliberate utilization of the ground water storage capacity as an addition to the volume of surface storage available in order to achieve maximum conservation of water supplies. Such a concept will result in the accomplishment of conjunctive use of surface and ground water storage.

The yield derived from a conjunctive operation is the total yield resulting from the operation of both surface and underground storage capacity. The proportionate part derived from each source will vary annually and cyclically. The quantity of water pumped from the underground basin must, in order to utilize the basin conjunctively, conform to the first criterion listed above. Criterion No. 2 must also be observed in order that the water thus made available is usable for beneficial purposes. The implication of single agency operation inherent in conjunctive use, however, relegates criterion No. 3 to a position of subsidiary concern.

1. Senior Hydr. Eng., State Div. of Water Resources, Sacramento, Calif.

It would be desirable to denote the yield from conjunctive operation by the use of a new term. The writer suggests the concept of "average operating yield" to describe the water supply thus made available. The annual extraction will vary from zero in a year of ample surface supply to a maximum in a year of grossly deficient surface supply. Determination of the quantity of average operating yield can be made as a result of an operations study covering a complete cycle of water supply, as in the case of a surface reservoir. The basin investigation required as a preliminary to such study, however, is far more complex than that necessary to determine the yield of a surface development.

The investigation of physical situations which are susceptible of being utilized in conjunctive operation will generally be conducted along three broad phases of inquiry. The first phase of investigation may be termed the geological phase, in which the conditions to be encountered on and under the surface are determined. The primary requisite is the location of underground storage space, which may be found in those geological formations that contain voids in sufficient volume to hold, transmit, and release water in the quantities necessary for the satisfaction of requirements. Following the determination that usable storage space for practicable operation exists, the porosity, transmissibility, chemical composition, location of intake areas, and other basic physical characteristics pertaining to hydrologic and hydraulic operations are ascertained.

The second field of investigation of possible subsurface storage areas is the hydrologic phase. This is concerned with determinations of the water supply available for the project; the regimen of its occurrence; the quality of the native, imported, and subsurface waters available; the operation of surface facilities for storing, transporting, and percolating surplus in subsurface storage; and innumerable additional factors bearing on the problems of routing the water supplies between the place of origin and the place of final disposal.

The economic phase is the third broad field of inquiry. This phase is concerned with the determination of places and amounts of use. It involves the classification of land areas as to suitability for irrigated agriculture, domestic, industrial, or other uses; determination of water requirements for the various uses; production, distribution, salvage, and disposal of water supplies made available, and other necessary studies. It may be noted that the grouping of phases of an operating study is made solely for convenience in illustration and bears no relationship to the order of work or usual concept of fields of endeavor. The engineer and geologist will function in close coordination throughout the investigation, and will be assisted, as required, by other concerned specialists.

From the basic data collected, the limits of variability in factors entering into the determination of yield can be established and the operating studies can be completed. Such studies must commence with the disposal of the yield from the surface reservoir. This may be distributed in many ways. A portion will be dissipated in surface evaporation; other amounts will be released to stream channels, to irrigation and domestic or municipal service; to direct percolation to underground basins; and some may be exported permanently from the area. Items of accretion to ground water storage are the deep percolation of water applied for irrigation, whether from surface or underground sources; the deep percolation of precipitation on overlying lands and of a portion of the water delivered for urban uses; the direct ground water recharge

from surface water supplies; seepage from natural and artificial water channels; and subsurface inflow from adjacent areas. Depletion of ground water supplies is caused by extractions for service of required water supplies to areas of use; rising water; uneconomic consumptive use in high water table areas; extractions for purposes of salt balance; and subsurface outflow from the basin. From such studies, made with varying operating criteria, the average operating yield and the volume of underground storage capacity utilized may be determined.

The portion of the total underground storage capacity that is shown to be capable of being dewatered during periods of deficient surface supply and resaturated during periods of above-normal supply may be considered as the usable storage capacity. Obviously, the volume of the usable capacity is limited by the capacity of the basin to yield or absorb water at predetermined rates sufficient for cyclic operation. Criterion No. 2, pertaining to water quality is applicable to this situation. Water quality factors may reduce the volume of gross storage capacity that could otherwise be classed as usable.

Maintenance of suitable quality of water supplies, particularly those intended to be used consumptively by irrigated agriculture, requires the consideration of the salt balance involved in the use and re-use of such supplies. The solution involves induced drainage of water from the ground water basin in amounts sufficient to maintain satisfactory mineral quality therein. The amount of water so drained away will constitute a future demand on the developed water supply. Under natural conditions, most ground water basins tend to fill with water and to overflow in the lower portions, thereby flushing out soluble salts contained in water originating on the tributary watershed and overlying lands. When aquifers in the basin are tapped by wells, the pumping draft lowers ground water levels to such an extent that in many cases the natural flushing of the basins ceases. Since the pumped water is largely used on overlying lands, soluble salts accumulate within the basin and tend to degrade the quality of the ground water in storage. If the situation is such that no discharge of water from the area, either surface or subsurface, occurs in the course of time, the concentration of salt compounds in the remaining water will become so great as to inhibit its use as a source of water supply. For this reason, the planned extraction and export of the quantity of water necessary for maintenance of adequate quality is required.

From the procedures outlined above, the total firm yield of the conjunctive operation and the average operating yield of the underground reservoir can be determined. The average operating yield will vary, of course, in accordance with the method of operation selected for the surface component, that is "carryover" or "fill and draw" operation. The disposal of such average operating yield will determine the amount of firm water supplies from the conjunctive operation which can be made available to beneficial uses.

Dr. Clendenen very properly pointed out that conjunctive operation of surface and ground water storage capacity is presently being practised in several areas of deficient water supplies in California. In other areas, an involuntary, or unplanned, operation has been in progress for many years. This is particularly true in extensive irrigated areas where a considerable portion of the applied water, from whatever source, finds its way to the main body of underground water in storage.

The writer cannot agree with Dr. Clendenen as to the basis of financing future water supply developments. Supplies which have been developed in the past were generally those which were well within the ability of water users to

repay. The history of irrigation development in the West, however, is replete with instances of developments at excessive cost, followed by the dishonoring of bonds and other securities and the dismemberment or reorganization and refinancing of districts and other water agencies. In view of the fact that much of the "cream of the crop" of possible water supply developments has already been developed, it is apparent that projects which are uneconomic in a repayment sense will necessarily have to be constructed in order to meet increasing demands for water. While much of the cost of such projects should properly be paid by the principal beneficiaries, the water users, the writer feels that support from state tax revenues is both required and proper in order to continue the maintenance of a modern, dynamic economy.

In the administration and operation of ground water storage capacity for conservation and use of available water supplies, a new field of operational problems will necessitate major activity toward their successful solution. These problems may be divided into broad areas, tentatively designated as legal, hydrologic, management, and operational. The problems in each area will, of necessity, contain elements common to one or more of the other areas, as the interrelationship is such that precise area boundaries cannot be drawn.

The present paper has, it is hoped, served to initiate effort in this regard. Engineers, particularly those engaged in water resource activities, should take a position of leadership in the development of a sound foundation of theory and practice for the maximum ultimate utilization of all available resources for the conservation of our national water supplies.

Discussion of "MEASUREMENT OF CANAL SEEPAGE"

by A. R. Robinson and Carl Rohwer
(Proc. Paper 728)

A. R. ROBINSON,[1] J.M. ASCE, and CARL ROHWER,[2] M. ASCE.—Seepage measurements, even when made under carefully controlled conditions, yield rates which are correct only at a specific place at the time of the tests. It does not necessarily follow that the same results should be obtained in the same area at a different time under different conditions. As a result, widely divergent results are frequently obtained when seepage measurements extend over a considerable period of time. This fact makes the solution of the seepage problem especially difficult.

Mr. Raymond A. Hill, M. ASCE, has taken exception to the total seepage losses reported by the writers for the 17 Western States, because more water is delivered to farmers than is ordinarily shown by the records. This is probably true, but it should be pointed out that more water is frequently available for delivery than is shown by the diversions from streams and reservoirs at the canal intake, because many canals receive waste water from higher irrigated land and intercept run-off from rainfall. This inflow cannot be accurately measured and although it is probably less than the amount of the excess deliveries to farmers, it is a compensating factor. It should be pointed out also that water records of the Turlock Irrigation District, California indicate a loss of water from diversion to irrigator of 27.2%. This record is for an irrigated area of 168,000 acres and is an average loss for the five-year period of 1950-1954. Whether the seepage losses for the 17 Western States given in the paper are correct or not, is probably not important because they were cited merely to show the magnitude of the seepage problem.

The comments by Mr. Dean C. Muckel, based on his extensive experience in ground-water recharge by water spreading, show that the difficulties encountered by the writers in the study of seepage are not unique. With reference to Mr. Muckel's question as to the effect of operating the inner ring when no water was in the buffer ring, this condition was not investigated by the writers. Mr. Muckel found that gases, other than air, were released in the soil under certain conditions. When this occurs, the seepage rate would be reduced.

The fact that the depth of water in the rings is not the true head was recognized in the study of effect of depth of water on the seepage rate. By projecting the curves downward until they intersect the vertical axis, a head is obtained which makes the seepage rate directly proportional to the depth, as should be the case according to Darcy's law. Since the seepage rings were used primarily for the purpose of checking seepage meters, a single installation was used in each type of soil. However, many seepage meter

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1. Agricultural Engr., U. S. Dept. of Agriculture, Irrig. and Drainage Investigations, Colo. A & M. College, Fort Collins, Colo.
2. Fort Collins, Colo.

measurements were made in each seepage ring. Observations of seepage were made at intervals extending over a period of about two weeks at each setting of the seepage meter. At the end of the period the seepage meter was installed at a new location inside the ring and the procedure repeated. As a result of this procedure, many replications of seepage meter measurements in different soils were obtained where the seepage rate was accurately known.

An ingenious method of keeping the pressure inside the seepage meter equal to the head on the canal bed is described by Mr. C. C. Warnick, J. M. ASCE. By using the Mariotte siphon principle, the reservoir of water for the seepage water can be placed above the water surface in the canal. This equipment permits the observer to see whether the seepage meter is functioning properly and to note the rate at which water is seeping away through the cup of the meter. Mariotte controls were tried by the writers on well permeameters but the equipment did not prove satisfactory because it was affected by temperature. Furthermore, the surface tension at the bottom end of the air inlet tube affected the sensitivity of the device, because the water level had to drop appreciably before the air bubble would break loose from the tube. A simple float control was found to be more sensitive and it was not affected by temperature.

It is difficult to understand why the plastic seepage bag is not capable of transmitting pressure at the same magnitude as that due to the water surrounding the bag as pointed out by Mr. Warnick. The writers had occasion to try different measuring devices on the meters at the same installations without finding any difference in the measured rate. The burette on the Weber Basin seepage meter is considered by many technicians as being too fragile for field use.

The tests, extending over a period of three years, which were reported by Mr. Warnick, showed that the losses measured by the seepage meter were 57, 67 and 69 per cent of the losses measured by ponding. The writers found, however, that seepage meters installed in the seepage rings generally indicated a higher rather than a lower rate. This difference is probably due to the fact that all the seepage from the seepage rings had to pass through the bottom, whereas, in the experiments reported by Mr. Warnick, a large portion of the seepage probably occurred through the side slopes of the canals where it is difficult to install the seepage meters.

The development by Professor Szalay is interesting, but several inconsistencies should be pointed out. Part of these discrepancies may be due to errors in translation. The true infiltration velocity is:

$$v_i = v_s/n$$

where n is the porosity and v_s is the bulk velocity. The interpretation given by Mr. Szalay applies only to the case of flow through a horizontal section where the gravitational forces would be constant. For the case of a constant depth of water on a horizontal soil surface the hydraulic head would be h + z, so that Darcy's equation would be:

$$v_s = K \frac{h + z}{z}$$

In terms of infiltration velocity through the pores:

$$v_i = \frac{k}{n} \frac{h + z}{z}$$

and

$$v_i = \frac{dz}{dt}$$

By eliminating v_i from these equations, the basic differential equation results:

$$\frac{zdz}{h + z} = \frac{K}{n} dt$$

Integration of this equation results in

$$z = \frac{Kt}{n} + h \log \frac{h + z}{h}$$

In view of the foregoing analysis, it is difficult to understand how Mr. Szalay determined the relationship which he has tabulated from data on infiltration.

The possibility of utilizing permeability measurements on disturbed samples of bed material in determining seepage losses, is mentioned by Dr. Lauritzen. It is believed that this method would not be an indication of a true seepage rate from a canal owing to the difference in seepage rates on the sides and bottom. In many cases the sides have a higher seepage rate due to stratification and secondary structure, i.e. cracks, root channels, and holes dug by rodents. The effect of these factors would be eliminated in disturbed soil samples.

In order for the greater hydration of clay minerals at higher temperature to be a factor in a reduction of permeability at these temperatures, the process would have to be reversible. Referring to Figure 12, it is noted that there is a cyclic variation so that when the water temperature is decreasing, the seepage rate is increasing.

The comments of the engineers who have contributed discussions of the paper, have emphasized the complexities of the seepage problem. Many uncertainties still exist, and because of the importance of seepage, the study of the problem should be continued.

DISCUSSION

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By substituting [illegible] from these equations, the basic [illegible]

[illegible]

[illegible] It is believed that this method would not be an [illegible] enough [illegible] to the difference in [illegible] and [illegible] the [illegible] have a higher [illegible] stabilization [illegible] secondary structure, i.e., cracks [illegible] The effect of these factors would be [illegible] and [illegible].

[illegible] for the [illegible] of clay [illegible] in a [illegible] of permeability [illegible] would have [illegible] 12, it is seen that there is a cyclic variation so that when the [illegible] temperature is [illegible] the average rate of [illegible].

The comments of the engineers who have constructed [illegible] of the paper, have [illegible] the [illegible] of the shrinkage problem. [illegible] and because of the importance of [illegible], the problem should be continued.

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Discussion of
"RIVERBED DEGRADATION BELOW LARGE CAPACITY RESERVOIRS"

by M. Gamal Mostafa
(Proc. Paper 788)

M. GAMAL MOSTAFA,[1] A.M., ASCE.—Many thanks are due the engineers who have given the profession some of their time and knowledge by contributing their discussions of the writer's paper. The methods suggested in the paper for the computation of riverbed degradation were based upon the assumption of ideal conditions which may in some cases be far from reality. However, as Professor Albertson and Mr. Liu have so properly stated "it is only through a simplified beginning that a more exact (and perhaps more complex) solution may eventually be found." Starting with the simplified case, and introducing the effect of natural conditions in a manner also as simplified as practicable, one can probably arrive at a reasonable procedure for the computation of the rate and extent of degradation. Such procedure may vary for different cases depending upon the natural conditions and the existing man-made river structures. For instance, it may be possible in some cases to assume that a river stretch between two control stations is straight and that its cross-section is constant. In other cases, the effect of every sharp turn may have to be studied separately. Irregularity in the river section may decide the beginning and length of steps in the step trial method of calculation.

The use of a constant discharge for a river reach in the analysis was by no means a representation of mean discharge as was interpreted by Messrs. Borland and Miller. It is true that natural stream flow should be represented by a flow-duration curve, but once a large capacity reservoir is constructed with the usual intention of regulating the flow into its downstream, it is very likely that the variation in discharge would be seasonal rather than an every day natural phenomena. Computations based upon a constant discharge would therefore be close to actual conditions. An example is the Aswan Sadd-el-Aali (High Dam) which will be constructed on the Nile with a reservoir capacity of 130000 million cubic metres for flood protection, generation of power and development of agriculture. Only water requirements for Egypt's irrigation and domestic use will be released from the reservoir. As a result, it is expected that discharge variation will be seasonal with a maximum of 240 million cubic metres per day and a minimum of 100 million cubic metres per day.

Assuming a regime of natural equilibrium had been established in the Colorado River prior to closure of Hoover Dam, valleys were merely areas where deposition followed by scour took place without much overyear change in levels. After closure of the Hoover Dam, degradation of the riverbed started below the dam and progressed downstream. By 1938 the exposure of rock ledges, gravel and coarse sands had stopped the process in some close-by reaches, but in some far reaches the bed surface was still composed of

1. Chf. Engr., Sadd-el-Aali (High Dam) Dept., Cairo, Egypt.

fine material. Degradation had been retarded but was still acting. That the backwater effect of Lake Havasu of Parker Dam did not extend beyond the head of the Mojave Valley does not mean that if Parker and Davis were not constructed at all, degradation would not have possibly been still occurring even below the Mojave Valley.

The writer agrees with Messrs. Borland and Miller that natural stream channels are heterogeneous and that an assumption of pure homogeneous channel can never be close to reality. However, the theoretical limit for degradation below a large reservoir is the ocean when no other dam or natural deposition trap exists in its downstream. In many cases, this trap is located just at the stream end as it approaches the sea in the form of a delta.

Lately, the writer has been carrying out some laboratory tests at the Hydraulic Research Station, Barrage, Egypt, in order to determine the characteristics of the bed armor after scour. Although these tests are still in progress, yet the following preliminary conclusions can so far be derived: (1) That coarse material of about 5% of bed composition is enough to form an armor which would prevent excessive local scour by forces just less than the critical force for its general movement, (2) That the limit of degradation of a channel bed may not only be the formation of the armor but may include a minimum for the surface slope. This conclusion is not yet quantitatively described. It is hoped that experiments in the 100 metres long and 1.5 metres wide experimental channel which will be soon started will aid in the definition of the limiting slope, and (3) That the diameter at 75% finer as a relative index of stability needs more experimental verification. Professor E. W. Lane[11] has recommended this size for stable channel design.

So far, experiments have shown that 2% coarse material is not enough to form an armor at an early stage of degradation; perhaps 4 or 5% of coarse material will do.

Uniformly graded bed material was sorted during a degradation experimental run such that the final top layer after equilibrium was of a composition varying with the depth and location of the sample and had a maximum sample mean size corresponding to 85% finer of the original bed material.

Prof. Albertson and Mr. Liu have recommended the application of the equation $u = \sqrt{\tau / \rho}\ \ \phi\ (Re/ks/R)$ for the velocity of flow (where $Re = uR/\nu$) in the steady uniform condition, which seems reasonable once ϕ is defined. Prof. Albertson has shown the writer during a visit to Fort Collins in 1955, some of his work in defining this function which was very promising. Dr. Leliavsky suggests the use of the Beleida formula for the Nile velocity of flow:

$$u = (147 + 3.92\ (Z - 10)^{0.383})\ R^{0.85}\ S^{0.72}$$

where Z = the mean silt in suspension in grams/met^3. However, its application in the calculation of degradation is hardly possible, since Z should vary for every step and time intervals.

Coefficient k in eq. (5) of the paper is much more reliable than the Manning n. Application of Eq. (5) to Nile data has shown a maximum error of 35% while the Manning equation has shown errors of up to 200%.

As for the sediment load, the choice of Straub's equation was for its simplicity and its applicability to bed loads of fine and medium sands, which is usually the case in degradation. The modified Einstein formula,[12] Meyer-Peter formula[13] and Kalinske's formula[14] may all be applicable to a wider

range of sediment sizes than Straub's formula. A very interesting reference for different studies in this respect has been given by Ning Chien.[15] Actually, none of these formulas can be called a complete scientific solution. The fact remains that hydrosediment science is still staggering. Brook's[16] findings in his thesis work at California Institute of Technology may as well add more confusion in the choice of a sediment transportation formula.

Yet, hydraulic characteristics of a stream flow should be interrelated with the sediment movement by the flow whether this sediment is totally in suspension, mostly on the stream bed or distributed over the water section. The effect of sediment movement on the velocity of flow should be included in any equation describing the sediment load. This, at present, shows in coefficients which may be given as functions of numbers (Reynolds and Froudes) representing flow conditions. A purely rational solution may as yet be found that will describe accurately by means of measurable factors the flow of sediment in a stream. It is towards this objective that many interested engineers throughout the world are still working.

REFERENCES

11. "Design of Stable Channels" by E. W. Lane, Transactions ASCE 1955.

12. "Computations of Total Sediment Discharge-Nebrara River near Cody, Nebraska" by B. R. Colby and C. H. Hembree, Geological Survey, Water Supply Paper 1357.

13. "Formulas for Bed-Load Transport" by E. Meyer-Peter and R. Mullar, International Association for Hydraulic Research, Third meeting, Stockholm 1948.

14. "Movement of Sediment as Bed Load in Rivers" by A. A. Kaliniske, Trans. A. G. U., 1947.

15. "The Present Status of Research on Sediment Transport" by Ning Chien, Proc. separate 565 ASCE 1955.

16. "Mechanics of Streams with Movable Beds of Fine Sand" by N. H. Brooks, Proc. separate 668, ASCE 1955.

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REFERENCES

11. "Design of Stable Channels" by E. W. Lane, Transactions ASCE, 1955.

12. [illegible]

13. [illegible]

14. [illegible]

15. [illegible]

16. [illegible]

Discussion of "METHODS OF DETERMINING CONSUMPTIVE USE OF WATER IN IRRIGATION"

by R. D. Goodrich
(Proc. Paper 884)

D. M. FORESTER,[1] M. ASCE.—In his paper, Mr. Goodrich presents an excellent resume of many of the studies on the so-called "duty of water" in irrigation. The many formulae and methods which have been developed emphasizes the interest of the engineering profession in developing a theoretical means of prior determination of the amount of water required for irrigation needs. The majority of the studies cited in Mr. Goodrich's paper deal principally with the water requirement of the cultivated plant under various conditions of temperature, sunshine, rainfall, and other climatic factors. Perhaps it is possible to arrive at an empirical formula which, when properly applied, will give the correct answer for the previous irrigation season. There are many, many methods or formulae which supposedly permit a fine determination of the consumptive use[2] of water in irrigation; all give comparable results.

The engineer, who has the problem of designing an irrigation development, finds in most instances that complete information on the water requirement is lacking. This is especially true for the semiarid regions of the western United States. Consumptive use determinations are only a part of his problem in arriving at the amount of water necessary for a successful development. The total requirement includes, in addition to the consumptive use, evaporation, deep percolation loss, transmission losses, probable rainfall and degree of effectiveness, and irrigator efficiency and conservativeness.

The determination of present "man-made" depletions in the Upper Colorado River Basin is, without question, a vast, complex problem. To arrive at the ultimate "man-made" depletion after partial or full development as presently envisioned by the Commission is even far more complex. Consumptive use depletion studies based on anticipated various crops and native vegetation may and may not hold true for more than one, two, or three decades. Then, too, there will be many other influencing factors which must be evaluated and applied. All this introduces the primary problem and, that is, where is the "stopping place" for refinement of the "arithmetic" and the "judgment" factors.

The Bureau of Reclamation has adopted and uses the Lowry-Johnson Method[3] of determining consumptive use in planning irrigation developments

1. Gen. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Cody, Wyo.
2. Defined as "the quantity of water, in acre-feet per cropped acre per year, absorbed by the crop and transferred or used directly in the building of plant tissue, together with that evaporated from the crop-producing land." (Consumptive Use of Water in Irrigation—Trans. ASCE, Vol. 94, 1930, p. 1351).
3. "Consumptive Use of Water for Agriculture" by Robert L. Lowry and Arthur F. Johnson, Trans. ASCE, Vol. 107 (1942), p. 1243.

or projects. The Lowry-Johnson Method, which is one of the "inflow-outflow" procedures referred to by Mr. Goodrich, has theoretical shortcomings, but the consumptive use determination by this method compares favorably with that found by some of the more recently developed methods. In the design of an irrigation system the total and peak water requirements are of primary concern and the "consumptive use" is only one feature—must the designer arithmetically refine his consumptive use requirements to the nearest tenth of an inch or one-hundredth of a foot when he cannot determine his transmission and farm waste loss much closer than 5-10-15 or 20 percent, or more, of the diversion? Some methods of determining consumptive use place great weight on so-called "effective heat units" during an empirical growing season based on temperatures and the percent of daylight hours. It is known that humidity has much to do with the actual consumptive use of the plant. Also, the latitude and elevation in which they are grown. It is generally recognized by designing engineers that the "demand" requirement in high altitude developments is greater than in areas of lower altitude although the temperatures are the same or higher in the lower areas. This factor could possibly, for refinement, require studies of light intensities and the relative effect on cultivated crops.

Many of the empirical methods which have been developed for the determination of "consumptive use" entail long, laborious determinations, the results of which, at most, can be no more than an approximated estimate. Although the so-called Lowry-Johnson Method is somewhat less involved than many others, it, too, approaches a degree of refinement in computation which may be questioned as to realistic practicability. Some of the procedures of this method which are open to question are:

A) The procedure outlined to determine the limits of the growing season. A more realistic approach would have been to use the running average of the consecutive daily minimum temperatures. The element of difference in the two methods of determination is insignificant.

B) The Lowry-Johnson curve reflecting the relation of consumptive use to effective heat has without doubt proven extremely helpful to the designer of irrigation projects although there are competent grounds on which one can question its correctness. As an example, the curve was determined on the basis of the average of determinations made for each of some 20 locations. However, in several instances the basic data were arbitrarily "corrected" by the authors. The curves developed by Messrs. Lowry and Johnson does not correctly reflect the actual basic data. Their curve can be competently expressed by the equation

$$H = 6500\ U - 5600$$

where:

H = Effective heat, in thousand day-degrees
U = Consumptive use of water in feet depth

By computation, using only the basic data and discarding the authors' corrected adjustments, the relation would be expressed by the equation

$$H = 5500\ U - 3100$$

The modified curve as expressed by the computed formula appears to

overcome some of the criticism found in several of the discussions of the Lowry-Johnson paper.

In general, the distribution of the irrigation supply, based on the total diversion, ranges from 30 to 40 percent, or more, for canal waste and losses and 35 to 45 percent loss for farm waste on unlined or partially lined gravity system. Thus, it is evident that extreme arithmetical refinement in consumptive use requirements is difficult to justify in view of the necessary approximated values used for canal and farm wastes and losses. These observations, although made specifically in regards the Lowry-Johnson Method, are also generally applicable to virtually all the many methods outlined in Mr. Goodrich's paper. From the practical viewpoint of good, sound engineering judgment are the extreme refinements used in determining approximate consumptive use values justified when designing a typical gravity irrigation system, large or small?

Discussion of "COST-ALLOCATION FOR MULTI-PURPOSE WATER PROJECTS"

by N. B. Bennett, Jr.
(Proc. Paper 961)

FREDERICK L. HOTES,[1] A.M. ASCE.—Cost allocation methods for water resources development projects have been required in the past primarily for use in the analysis of proposed Federal works. With the entrance of state governments into the active financing, construction and operation of multipurpose water projects, it can be expected that cost allocation will be necessary in this area also. Furthermore, the growing participation of Federal monies in projects built by local districts makes it imperative that all engineers in water development know the principles and details of Federal cost allocating methods. It is now almost axiomatic that any flood control benefits that might accrue from a local project will be presented before the Congress with a request for Federal financing on a non-reimbursable basis. Other non-reimbursable benefits may, on occasion, also be claimed. While Congress in principle might look favorably on such requests, it will certainly require details to be submitted as to the benefits derived, and the proper allocation of costs, undoubtedly in accordance with Federal practices. Federal financing of "small projects" will also require the use of some accepted method of cost allocation. The author has, therefore, performed a distinct service in reviewing the most current practices in this regard.

While there are many methods of cost allocation, they all share a common intent—that of attempting to distribute the total costs of the project equitably among the various functions, or beneficial effects, of the project. This simple aim is amazingly difficult to achieve, as the author has so ably demonstrated by presenting the results of some of his experiences with the Separable Costs —Remaining Benefits method of cost allocation; presently one of the better available techniques in this field.

The author has indicated, at the bottom of Page 961-7, that the method does not always yield equitable results. The writer would like to pursue this thought by discussing a limitation on Item 3, Page 961-3, of the broad principles set forth there, towit, that "The maximum allocation to each purpose is its benefits or alternative single-purpose cost, whichever is less."

The writer recognizes the danger in using hypothetical examples, but believes that the following illustration of the use of the method will demonstrate a definite limitation for certain instances.

A certain project has only irrigation and power benefits. The total project benefits are 58 million dollars and exceed the total project costs of 56 million dollars. At this stage of the analysis the project analyst does not know whether or not each purpose will yield benefits greater than cost, since he does not know what proportion of the cost should be allocated to each function. Following the outline of the author, the costs are allocated as in Table I.

1. Associate Prof. of Civ. and Irrig. Eng., Univ. of California, Berkeley, Calif.

TABLE I.

Hypothetical Application of
Separable Cost-Remaining Benefits Method
Revealing Uneconomic Purpose
(Thousands of Dollars)

Item	Project Purpose: Irrigation	Power	Totals
1. Benefits	$23,000	$35,000	$58,000
2. Alternative single-purpose cost	27,000	30,000	
A. Lesser of (1) or (2)	23,000	30,000	
3. Separable Costs	8,000	29,000	37,000
4. Remaining Benefits (2A-3)	15,000	1,000	16,000
Percentage of Total	94	6	
5. Unallocated Joint Cost (Project Cost-Σ3)			19,000
6. Allocated Joint Cost	17,800	1,200	19,000
7. Total Allocation (6 + 3)	25,800	30,200	56,000

The analysis now reveals that the irrigation costs exceed the benefits. The proper conclusion, of course, is that the inclusion of irrigation under these circumstances is uneconomical and unjustified. It would not be right to apply Item 3, page 961-3, and limit the cost to benefits. In addition, inclusion of the irrigation portion in this instance yields a power assessment cost that is greater than the cost of an alternative project—also an uneconomical proposal.

It would appear then that a proper and necessary extension of Item 3 of the principles of cost allocation set forth by the author would be, that if the allocation costs for any particular function exceed the corresponding benefits, then that function should be removed from the project, or the project redesigned; rather than to reduce the allocated costs to the level of the benefits. This is in full accord with the criteria set forth in the Report to the Federal Inter-Agency River Basin Committee,[2] wherein it is required that "each separable segment or purpose provides benefits at least equal to its costs.

Another limitation of the Separable Costs—Remaining Benefits method is illustrated by the analysis of Table II, for the case of a reservoir designed entirely for irrigation yield, but which does inherently provide some recreation benefits. Both the separable and the specific costs for recreation are assumed to be zero, i.e.—the recreation benefits accrue even though no specific part of the structure was designed to solely provide these benefits. It is logical and fair that the recreation function be allocated some share of the cost, as some benefit is gained. Perhaps the best method of allocation in this case would be by proportional benefits, i.e. have the total cost allocated in proportion to the benefits derived.

Whereas the proportional benefits method is equitable in the case of Table II, it is not correct to use it in the first case shown in Table I. Indeed,

2. "Proposed Practices for Economic Analysis of River Basic Projects," Report to the Federal Inter-Agency River Basin Committee, prepared by the Subcommittee on Benefits and Costs, May 1950, p. 37.

TABLE II

Hypothetical Application of
Separable Cost-Remaining Benefits Method
Illustrating Limitation for Case when Separable Cost is Zero

Item	Project Purpose: Irrigation	Project Purpose: Recreation	Totals
1. Benefits	$23,000	$2,000	$25,000
2. Alternative single-purpose cost	23,000	5,000	
A. Lessor of (1) or (2)	23,000	2,000	
3. Separable Costs	20,000	0	20,000
4. Remaining Benefits (2A-3)	3,000	2,000	5,000
Percentage of Total	60	40	
5. Unallocated Joint Cost (Project Cost-Σ3)			0
6. Allocated Joint Cost	0	0	0
7. Total Allocation (6 + 3)	20,000	0	20,000

one of the most vital weaknesses of the proportional benefits method is that it permits the inclusion of an uneconomical component as long as the total benefits exceed the costs. It is this very deficiency in this simplest of all approaches to cost allocation that compels the use of more complicated methods.

It is probably safe to say that as yet there is no single method of cost allocation that gives equitable results in all cases. The engineer and the economist must use the best of several methods, or a combination of several methods, to attain an equitable distribution of costs.

While the author has admirably eliminated details not germane to his basic thesis, there is one point that was mentioned that perhaps needs amplification. The author has indicated that, for the sake of simplicity, discussion of "taxes foregone" was omitted. However, under Step 7, Table I, Page 961-6, the following modifying phrase appears.

> ". . . . . . and, in the case of power, subtract from that sum the amount of taxes foregone which was used in computing power costs under Steps 2 and 3 above."

The writer was somewhat confused as to the full significance of this phrase. Apparently these "taxes foregone" would be included in Steps 2 and 3 only if the benefits in Step 1 included comparable taxes. The cost allocation process would yield approximately the same results if taxes were omitted from the first three steps. The inclusion or exclusion of taxes would probably depend directly on the most convenient method of determining the benefits, and indirectly on the policies of the agency conducting the study. Comments of the author in this regard would aid in clarifying this point.

WENDELL E. JOHNSON,* M. ASCE, and CHARLES A. COCKS.**—The problems involved in making an equitable distribution of costs among functions

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* Chief, Eng. Div., Missouri River Div., Corps of Engrs., U. S. Dept. of the Army.

** Chief, Planning Section, Missouri River Div., Corps of Engrs., U. S. Dept. of the Army.

served by multiple-purpose water projects are of considerable importance. As development of this Nation's water resources progresses, the multiple-purpose project is becoming more important. The author in attempting to rationalize the concepts appropriate for use in cost allocation studies is performing a very necessary service and is to be commended for his excellent paper. The exchange of ideas and theories, improvement in basic concepts and their use will lead to more uniform and more appropriate solution to the problems.

As stated by the author, the Separable Costs-Remaining Benefits method of cost allocation is relatively new, and, as such, precise understanding of its procedural details and basic concepts has not been fully achieved. The paper points out that the method depends upon benefits and alternate costs and discusses the meanings attached to single-purpose alternate costs, specific and separable costs, joint costs, and remaining project costs. In many respects the Separable Costs-Remaining Benefits method is closely allied to the alternate justifiable expenditure method of cost allocation. This latter method has been in use for some time, and procedures and policies relative to its use are quite well established. Thus, the factors common to both methods are well understood and well stated by Mr. Bennett in his paper.

As stated by the author, the only significant difference in the two methods is the use of separate costs in lieu of specific costs. With regard to specific costs, the author states, "The readily determinable costs of facilities which are clearly for one purpose only should be allocated specifically and wholly to that purpose." The author further states, "Separable costs include specific costs plus that portion of joint costs which is traceable solely and clearly to the inclusion of a single purpose in the multiple-purpose project." The report of the Subcommittee on Benefits and Costs of the Federal Inter-Agency River Basin Committee in "Proposed Practices for Economic Analysis of River Basin Projects" (frequently referred to as the "green book," dated May 1950), in which the Separable Costs-Remaining Benefits method was proposed, states "The separable cost for each project purpose is the difference between the cost of the multiple-purpose project and the cost of the project with the purpose omitted."

In explaining his concept of separable costs, the author states that it is an extension of the basic idea of direct assignment of specific costs; that separable costs are defined as the difference between the cost of the multiple-purpose project and the cost of the <u>same</u> project with the purpose omitted (underscoring supplied); that the remaining project need not be justified for cost allocation purposes; that the remaining project need not be the most economical source of providing multiple-purpose benefits to the remaining functions; that although the alternative single-purpose costs bring a comparison of plans of different nature or location into the cost allocation study, the remaining project should not; that the remaining project should provide benefits to each remaining purpose at least equal to or in excess of those obtained from the multiple-purpose project; and that the remaining project should not interfere in any way with or adjustments required in the remaining functions.

Some of these restrictions imposed by the author on the process of determining separable costs would appear to negate, in part at least, the theory of separable costs as indicated in the "green book." This report after defining separable costs as quoted above in the third paragraph, continues in explanation as follows: Separable costs include more than the direct or specific costs of physically identifiable facilities serving only one purpose, such as an

irrigation distribution system. They also include all added costs of increased size of structures and changes in design for a particular purpose over that required for all other purposes, such as the cost of increased reservoir storage capacity. In effect, separable costs are computed from a series of project cost estimates, each representing the multiple-purpose project with one purpose omitted. Such information will be readily available when the recommended practices of project formulation have been followed. Where project formulation has not been of the detail suggested in the recommended procedure, it may be necessary to use specific costs in lieu of separable costs in those cases where re-estimating would be unduly burdensome."

In discussing the application of procedures recommended in project formulation, the "green book" states "The next step in river basin study should be to examine and analyze the physical possibilities for improvement or development of the basin's resources to meet the needs or objectives. At all stages of such analysis, preliminary, intermediate, and final, the advantages and disadvantages of the various physical possibilities can and should be evaluated and compared in terms of benefits and costs, measured with successively increasing degrees of refinement, as required, to eliminate the obviously unjustified and least favorable possibilities, until the optimum plan of development is formulated." It further states, "At various stages of project formulation, the program, project, or segment of a project under consideration must satisfy the criterion that it would be more economical than any other actual or potential available means, public or private, of accomplishing the specific purpose involved. A program, project or segment of a project should not be undertaken if it would preclude development of any other means of accomplishing the same results at less cost. This limitation applies to alternative possibilities which would be displaced or economically precluded from development if the project is undertaken. Other means of obtaining similar benefits which would not be precluded from development are not limitations on project justification but are, in effect, additional projects which may be compared in an array to determine which should be given prior consideration from the standpoint of economic desirability." . . . "The alternative possibilities to be considered in applying this limitation should include all practicable means of accomplishing the desired results which are within the purview of the agency making the economic analysis. In theory, the broadest possible range of alternatives for any given objective should be considered but it is recognized that in practice, the range of alternatives that can be considered at regional levels may be limited by the information available at such levels. Also, there may be alternative possibilities which are not known to an agency responsible for project analysis. Nevertheless, consideration of alternatives on the broadest possible basis should be given at all levels of responsibility and necessary information for that purpose should be exchanged among the Federal agencies involved and utilized at appropriate levels of project analysis and review."

It thus appears evident to the writers that the Separable Costs-Remaining Benefits method as outlined in the "green book" was dependent upon the full process of project formulation in which the engineer had full freedom, in fact responsibility, to determine the most economical means of accomplishing the purposes involved including the determination of costs of inclusion of a function. The writers believe that the remaining project or projects should be formulated to serve the remaining functions by the most economical

source or sources of providing the same benefits as would be provided by the full multiple-purpose project, and that such remaining project or projects should be economically justified. The separable costs of a function would, therefore, be the difference in costs of the multiple-purpose project and the remaining project with the function omitted, with the remaining project adjusted as required to serve the remaining functions to an extent comparable with the multiple-purpose project. In this way the true net increase in the cost of including a function is obtained. If a remaining project, omitting one function, were allowed to provide beneficial effects significantly in excess of the multiple-purpose project it would be left to individual discretion to set the amount of such excess which would largely preclude the derivation of comparable estimates. To assure comparable estimates for remaining functions the service rendered by the remaining project should therefore be comparable to the service rendered by the multiple-purpose project. As stated by the author, this might involve the provision in the remaining project of facilities of lesser scope and cost that are specifically for a single purpose. This is not inconsistent in that it is a measure of the extra costs involved for that function to meet a specific need when the separable function is included. As stated in the "green book," during project formulation all possible alternatives should be investigated. This would involve in some instances the comparison of alternate locations for remaining projects to determine the most economical means of providing service.

It thus appears that the author has described a modification of the alternate justifiable method which would slightly extend the concept of specific costs rather than a clarification of what separable costs should be under the Separable Costs-Remaining Benefits method as proposed by the Subcommittee on Benefits and Costs of the Federal Inter-Agency River Basin Committee in their report, dated May 1950, entitled "Proposed Practices for Economic Analysis of River Basin Projects." It is hoped that the Inter-Agency Committee on Water Resources will take action to clarify this apparent difference in concept.

EUGENE W. WEBER,[1] M. ASCE.—The concepts and principles for application of the Separable Costs-Remaining Benefits method of cost allocation have been thoughtfully analyzed and effectively presented by the author.

Experience in the application of this method of cost allocation in the relatively few years since it was proposed by the Federal Inter-Agency Subcommittee on Benefits and Costs has indicated that it may be desirable to depart from the very rigid interpretation of separable costs as defined by the author on the basis of the original discussion of separable costs in the Benefit-Cost Subcommittee's report. Usually it will be possible to obtain satisfactory estimates of separable costs by computing the difference between the cost of a multi-purpose project and the cost of the <u>same</u> project with a purpose omitted. Occasionally, however, it may be necessary to take into account the fact that the most logical project for all purposes except the one for which separable costs are being computed is a project other than a variation of the multi-purpose project in question. When this is the case, the writer believes that a broader concept should be followed and that the separable cost for any purpose should be computed as the difference between the multi-purpose project

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1. Special Asst. to Asst. Chief of Engrs. for Civ. Works, Office of Chief of Engrs., Washington, D. C.

cost and the cost of the best available and most economical project that will afford equivalent results for all of the other purposes.

If this broader concept is not followed when there are in fact possibilities for more economical ways of achieving the desired results for all but one of the purposes, the objective of equitable sharing in the advantages of the multi-purpose project will not be fully realized in the cost allocation.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(ST), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

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c. Discussion of several papers, grouped by Divisions.

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